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June 6, 1940

Montana State Highway Department
Helena, Montana

Gentlemen:

I acknowledge receipt today of Proceedings of The Montana National Bituminous Conference, Sun Valley, Idaho, Sept. 11-14, 1939.

I am placing this volume in our Engineering Library; it is a valuable record.

With thanks and appreciation,

Truly yours,

C Derleth Jr.

C.Derleth,Jr.
Dean, College of Engineering

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The Montana National Bituminous Conference

SUN VALLEY, IDAHO

September 11, 12, 13, 14,

1939

PROCEEDINGS PUBLISHED BY THE STATE OF MONTANA
AND THE MONTANA STATE HIGHWAY COMMISSION

Montana Highway Commission

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LEE M. FORD, Chairman
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D. A. McKINNON, State Highway Engineer

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State Highway Engineer, Montana State Highway Department

General Chairman
1939 Montana National Bituminous Conference

D. L. CHENEY
Bituminous Engineer, Montana State Highway Department

Assistant General Chairman
1939 Montana National Bituminous Conference

H. R. FLINT
Director of Highways, Idaho Department of Public Works

Vice Chairman
1939 Montana National Bituminous Conference

OPENING SESSION

Montana National Bituminous Conference

SUN VALLEY, IDAHO

9:15 A. M.
September 11, 1939

D. A. McKINNON, State Highway Engineer
Montana Highway Department
General Chairman

D. L. CHENEY, Bituminous Engineer
Montana Highway Department
Assistant General Chairman

H. R. FLINT, Director of Highways
Idaho Department of Public Works
Vice Chairman

Opening Address

By

D. A. McKINNON
State Highway Engineer
Montana Highway Department

General Chairman, 1939 Montana National Bituminous Conference
In Behalf of Governor Roy E. Ayers, Governor of the State of Montana

Gentlemen, in opening the sessions of the Fifth Annual Montana National Bituminous Conference, I first wish to express on behalf of the Executive Committee our sincere appreciation of the splendid cooperation we have again received this year from the committee chairmen and the committee members.

During the coming three days you will all see the vast amount of work they have done during the past few months in preparing papers and assembling material for this meeting.

We who attend these meetings are all either engaged in or interested in all of the many phases of bituminous highway surfacing and consequently are united in a common effort to constantly improve our highway surfaces and at the same time wherever possible reduce their cost. I believe that through this Conference we have greatly assisted in making possible the rapid and remarkable progress which has been made during the past few years in the development and improvement of bituminous surfaces. This would not have been possible had it not been for the wholehearted cooperation of the men engaged in bituminous work throughout our country and Canada who have given freely of their time and knowledge to facilitate in any possible way the dissemination of bituminous knowledge.

We had hoped to have with us to welcome you this morning the Hon. Roy E. Ayers, Governor of our State, but illness has prevented his attendance and he has requested me to convey to you his regret at being unable to be here.

On behalf of our Governor and the Montana Highway Department I wish also to thank our hosts, the Idaho Highway Department, and the other people of Idaho who have assisted, for their splendid hospitality and cooperation. We had also hoped to have present this morning the Governor of Idaho, the Hon. C. A. Bottolfsen, in order that he might extend to you a welcome on behalf of the state of Idaho. However, it has become impossible for him to be here and in a few moments I am going to call upon Mr. H. R. Flint, Director of the Idaho Bureau of Highways, whom Governor Bottolfsen has delegated to represent him here this morning.

I am sorry to announce that we have two changes in our program. We have been advised that it will not be possible that Prof. Lang or Mr. Upham be with us. Prof. Lang could not attend due to a serious illness in his family and Mr. Upham had to cancel his trip out here as it was not possible that he leave his Washington office due to press of business. We regret very much that these gentlemen cannot be with us as they have devoted quite a lot of time and effort to the preparing of some very fine papers. However, Mr. H. H. Houk of the Civil Aeronautics Authority of Washington, D. C., has very kindly agreed to present Prof. Lang's paper and Mr. Upham has delegated Mr. W. C. Slee, Assistant Director of the American Road Builders' Association, to represent him and present his paper.

I now wish to introduce Mr. H. R. Flint, Director of the Idaho Bureau of Highways.

Address of Welcome

By

H. R. FLINT

Director of Highways

Idaho Department of Public Works

Vice Chairman, 1939 Montana National Bituminous Conference

This certainly is an unusual occasion when governors will forego an opportunity such as this and transfer it to the hands of those in the high organization.

I wish to express to you Governor Bottolfsen's regrets at not being able to be here. He had planned all summer that this would be the one occasion he thought he might be able to attend and, as he said, ''take his hair down.'' However, the press of executive business has made it impossible for him to be here and, as I have said, he asked that his regrets be expressed to the convention.

I am sorry he is not present, because he would add materially to the dignity. He is a man you would all like immediately. He is very attractive, has a wonderful personality, and an eye that sparkles at every little bit of amusement, and he makes friends most readily.

However, I do feel that something picked out of the borrow pit might be rather apt at this time. A negro was driving down the highway at a rather excessive rate of speed, and was stopped by the traffic officer and severely reprimanded for tearing up the road. He asked suddenly, ''Haven't you a governor on that car?''

''No, suh, boss, no ,suh; de governor am back in de state house. What yuh smells am fertilizer.''

I realize that a great many of you are from outside the State of Idaho and, like most of us, you probably are somewhat unfamiliar with Idaho's peculiarities, because I am basing that on our knowledge of other states. Idaho is somewhat unique in many respects. I throw out before you an aerial map of the state which is a picture of a relief map made to vertical and horizontal scales. You will notice the state is shaped something like a pork chop. In the north we find the bone; in the south we find the meat. Even the people are different in the separate parts of the state. In the north we find that they dig in the hills and live like moles and the farmers seek the highest elevations for their farms, and then we have to build a road up a rocky cliff for every farmer. In the south they put their agriculture where God intended it should be, where they can get water to it, and they live in the wide open spaces.

The timber is different. In the north we have the tall, white pine, and other pines of the lumber variety, while in the south the sagebrush and sage sticks predominate.

The matter of building our roads in the north is far more expensive than in the south, and we are continually hampered by trying to equitably divert our funds to provide the people with the necessities in the way of transportation.

We probably have more neighbors than any other state in the Union. If you will notice, Idaho is bounded by six states, plus Canada. While not the largest state in the Union, our peculiar political lines give us that distinctiveness.

We hope you are going to enjoy your sojourn in Idaho. We know that some of you came through by the Craters of the Moon, and undoubtedly were very much disappointed in the transportation facilities, at least in the way of highways, through there. We are concentrating on that particular section and hope within the next two or three years to have a well oiled highway all through that section, which will not only increase the popularity of the craters, but Sun Valley and primitive Idaho which lies to the north of this location. We trust your sojourn here will be so pleasant that you will hurry back and give us more time in the future.

Safety on the Highways

By

DR. RALPH L. LEE
Public Relations Staff
General Motors Corporation
Detroit, Michigan

In a little group having luncheon some time ago Sidney Williams, whom most people consider one of the best posted highway safety workers in the country, said that he believed he could tell just how long a person had been in safety work by the accident cures he suggested. If, Sidney went on to say, the person in question sprung a panacea right off the bat, and was extremely indignant over the dumbness, indifference and ineffectiveness of the officials in charge of highway safety, he knew for a certainty that he was facing a brand new recruit to the safety cause and one who just recently had discovered that there was such a thing as a highway safety problem. If the specimen did not spring a panecea, but felt that little had been done of a constructive nature, so far, and that the world was waiting for him to bend the old bean and come through with the answers, then, Sidney continued, this recruit, while not so embryonic as the panacear, was still wet behind the ears.

But if, on the other hand, the safety suspect had no panaceas or cure-alls to suggest, was sympatheticaly inclined toward other safety workers and officials and their problems, did not expect results too quickly, then, Sidney said, he was sure he had a dyed-in-the-wool safety worker who knew what he was talking about.

In looking back over my experience in safety work, I recognize the stages I have been passing through in the examples Sidney used to illustrate his safety worker analyzer. Now that he has pointed it out, I begin to feel that I, too, appreciate the fact that the more experience a safety worker has, the less latitude he feels he has to concoct surefire cure-alls, the less faith he has in panaceas, and the more respect he has for past experience in safety matters. With this respect goes a recognition that, regardless of the plan or program favored, highway safety is essentially a long haul project, for it deals primarily with human conduct.

Human beings, whether they be on foot, on wheels, or resting in chairs, behave pretty much alike, and have not changed greatly over long periods of time. When change is made, it invariably comes about as a result of sort of a wearing down process similar to that involved in learning to play the piano. For in learning to play the piano, knowing what is to be done is not enough. You've got to try to play, make mistakes, correct these mistakes and, above, all practice, more practice, and still more practice, until your fingers automatically do what your mind tells them they ought to do.

Regardless of the statistics which have been gathered together to rouse and stimulate us into action, accidents do not occur en masse, by the state, county, city or town, but singly, involving individuals — and individuals who are surprisingly like ourselves—who think and drive very much as we do. Until we get this picture, we are not qualified to think and act sanely with regards to highway accident prevention.

Therefore, it seems to me when we are confused or try to judge the merits of a suggested safety procedure, plan, or device, it would be well for us to back out of the maze of safety trees which obscures our view of the safety woods, and go to the source of the problem—the individual driver, pedestrian, policeman, judge or safety official who, in various combinations and under various circumstances, combine to cause and cure our collective accident problem.

A classification of accidents has occurred to me which on a number of occasions, has stopped me from running around in circles and getting lost in the safety woods. While I know that accidents have been classified to death in the past, I am going to run the danger of being classified myself, as still wet behind the ears, "safetically" speaking, and present this pet classification of mine for what it may be worth to you.

To my way of thinking, there are just three kinds of accidents:

1. Accidents caused from doing the things we already know are wrong.
2. Accidents caused from doing the things we did not know were wrong.
3. Accidents caused by conditions beyond our control.

In the use of the last item in our formula, I would strongly recommend that new recruits in the field of safety work handle it with kid gloves, for it is taboo and highly unethical in professional safety circles. For general public consumption and highway safety promotional purposes, we will surely lose cast if we even so much as hint that accidents happen and are not caused. But in spite of this, I am willing to run the danger of including accidents in my classification formula with the hope that I can justify myself later.

Let us take the first classification.

Accidents Caused by Doing the Things We Already Know are Wrong.

I have no statistical support in what I am going to say, but I firmly believe that most of our accidents come under this category. Why should human beings behave differently on wheels than they do on foot or while seated? For several thousand years—and long before the motor car was even thought of—priests have led their congregations in a confession which includes the following: "We have followed too much the devices and desires of our own hearts. We have offended against thy holy laws. We have left undone those things which we ought to have done; and we have done those things which we ought not to have done; and there is no health in us." Since we have quoted a general confession, we can rightfully assume that even the best of us, now and then, do the things we already know are wrong, and have been doing them for quite some time. If, as I guess, most of our highway accidents come about as a result of this almost universal human characteristic, then Highway Safety Educational Programs designed primarily to tell people what they already know are not only likely to be ineffective but are more than likely to bore our highway sinners. I'm afraid that the vast army of traffic sinners who pile up most of our highway accidents can only be reached through anticipation of a day of judgment and the fear of a little personal suffering as a consequence of their traffic sins.

Until the certainty of an inevitable judgment day is felt by the majority of the congregation of motorists and pedestrians who shuttle back and forth through the heavy traffic of a community, I'm afraid that sermons from the safety pulpits, no matter how eloquent and moving, are not going to do the safety trick. Plenty of uniformed traffic angels, on foot, on horseback, on motorcycles, and in scout cars have an astounding effect on the behavior of the congregation. In fact, you invariably see a wave of law observance preceding and following one of these angels whenever he moves among us.

While I love my fellow men and like to be considered his friend, I would rather be the kind of friend that Elbert Hubbard told about when he said: "A friend is a person who knows all about you—and still likes you." I don't think that any safety program which does not have as its very foundation just traffic laws, certain and impartial enforcement and unfixable traffic courts has the slightest chance of success of reducing highway traffic accidents. This is no new discovery—communities all over the country have proven this to be true. And the most encouraging thing of all is that we need not invent satisfactory laws, methods of enforcement, or honest cooperation by the courts—they have already been invented, tested, and proved; and the procedures for obtaining them are now available to any community sincerely interested in accident reduction and appreciation of the fact that most of our accidents come about as a result of individual people doing the things they know are wrong.

Ignorance of the law is never acceptable as an excuse for breaking the law, but we would be foolish indeed if we attempted, through law enforcement alone, to reduce accidents which come under our second classification:

Accidents Caused by Doing Things We Did Not Know Were Wrong

In spite of the fact that we have made this classification second, no safety program which does not have a friendly non-paternalistic and helpful educational activity can be wholly successful. However, it seems to me that the sifting process through which we separate the things most people know about driving, walking, and safety on the highways from the things they do not know, is the key to the effectiveness of such a program.

You may accuse me of overrating this sifting process and feel that whether people know what they ought to do in traffic or not, they cannot be hurt from being told what they already know, over and over again. And you may have a point, for, in spite of the fact that we all know what we ought to do in case of pyorrhea, halitosis, acidosis, athlete's foot, strained muscles, wrinkles, falling hair and hangovers, millions of dollars are spent every year for educational activities. However, unless we recognize symptoms of the ailments and feel the need for a cure, educational campaigns, regardless of the money spent on them, make very little impression upon us. Which convinces me that, from an effectiveness standpoint, it is dangerous to sandwich in among the many things people already know a few things they do not know—particularly so, when the average motorist and pedestrian is convinced that he is a good driver and walker, does not have traffic halitosis, pyorrhea or acidosis, and therefore, needs no remedy.

We must recognize that not only does the average person think he knows how to drive and avoid accident, but he actually does most of the time. Twenty-nine and one-half million motor cars each being driven about 9,000 miles a year provide us with the astronomical figure of 260,000,000,000 car miles. Divide this incomprehensible statistic by the 32,400 motor traffic fatalities—almost a third of which were pedestrians—and we are forced to the conclusion that an awful lot of driving is done for every death. In fact, 9,000,000 miles for each fatal accident. Divide the 260 billion car miles by the personal injury accidents and we find 300,000 miles are driven for each accident causing injury. Divide it any way you like, and answer will be the same—the average American driver does know how and does drive skilfully and safely most of the time.

While the reckless, lawless, drunken, and physically and mentally deficient driver and pedestrian is accountable for many more accidents than the average person, it is the average person who drives for three, four, five and ten years without an accident, who is responsible for over three quarters of all our accidents. Therefore, a highway safety program designed primarily to reach the ignorant, lawless and careless person will offend the usually careful driver, and even though it does a good job on those for whom it was intended, the effect on the overall problem will only be slight. The safety zealot who approaches the skillful and usually safe driver and walker on the basis that he is a moron has no concern for his own life or property, or the life and property of others, will not even get an audience—let alone wield an influence.

It begins to appear, therefore, that our highway safety educational plan cannot be too paternalistic, preachy, or too elementary, if we hope to affect the people causing the accidents under our second category, "Accidents Caused by Doing the Things We Did Not Know Were Wrong." We cannot approach these people in a corrective mood but rather as companions with driving and walking habits which also need correcting. But in spite of all the safe driving that is done, there are many tricks to safety which most people do not thoroughly understand. Taking curves, skidding, how to do it, avoid it, and recover from it. Behavior in a blow-out. Running off a shoulder and getting back. How and when to pass a car with safety. The scientific foundation for good trraffic laws. An insight into the problem of our traffic police, the timing of our traffic lights. The nonpreachy and friendly sharing of the why and wherefores of all that is being done to help people get where they are going as quickly and as safely as possible, must be the dominating note to our safety propaganda if it is to be accepted and put into practice.

Reckless and dangerous drivers must be sifted out from the multitude and given drastic treatment, but this same treatment will never work for the vast majority responsible for most of our accidents. Two prescriptions are necessary.

While I agree with our safety people, that in the main we must look upon all accidents as preventable, we must admit the fact that they are not all preventable as far as the individual is concerned. I am now talking about our third and last classification:

"Accidents Caused By Conditions Beyond Our Control"

We have learned that the physical or engineering aspects of the highway safety problem are extremely important. Our traffic and highway engineers have proved that they can design a certain amount of automatic safety into our streets and highways. And, some of our more ambitious engineers go so far as to say that with all the money they need, they can design and construct most of our accidents out of existence. Whether this be true or not we do know and have proved that many accidents are caused by physical defects in our street and highway safety systems. Little can be done in this direction without money and money cannot be secured without the public's understanding and appreciation of the extent to which its interests are affected by physical defects in its traffic channels. The engineering problem divides itself, so I am told, into two general classifications. Corrective changes in the existing set-up of streets and highways and planning and reconstruction for the future. Somewhere between the extremists and the ultra-conservatives is the point where a community must begin its physical highway safety activities. One thing is certain—money is necessary. Economics is just as important to the safety movement as it is in any other human activity. And again I am happy to say that sufficient pioneering has been done to point the way to any community in establishing a practical safety engineering program in keeping with its financial status, its needs, and peculiarities.

I am glad that Sidney Williams is not in the audience for I am afraid that he would have me vivisected, analyzed and diagnosed as a highway safety guinea pig and definitely catalogued as extremely wet behind the ears. However, I am very firm in my conviction of the following:

1. Without any new inventions or discoveries in the safety field, enough information is already known, proved and

available with which any community, whether it be a state, a county, a town, or a city, can apply to its highway safety safety problem and definitely reduce accidents.

2. The highway safety job must be done by the safety officials chosen by the public to do the safety job, and while they cannot do this important job without the support of the public, neither can the public or any group, regardless of their sincerity or zeal, help with the work except through the official safety people.

3. Accident prevention involves the changing of human beings. This takes time ,therefore, spasmodic campaigning—regardless of its intensity—will have little effect on the safety problem in the long run. Sane, steady, consistent, continuous cooperative effort has proved to be the only means of solving the highway safety problem.

4. Regardless of proven plans—or experienced personnel—nothing can be done without the support of the rank and file citizen, and his support cannot be secured in a haphazard or hit and miss fashion. Carefully planned public relations activities are just as important as any of the three E's.

5. Nothing can be done without money. But again we have proven that money can be secured when we have sold the public on the profit each individual will realize in terms of safety and satisfaction from a public investment in safety activities. I still think that my pet classification of accidents will guide us in the emphasis we place upon the many activities which must coordinate to solve our highway safety problem.

The Importance of the Design, Construction and Maintenance of Bituminous Surfaces to National Highway Safety

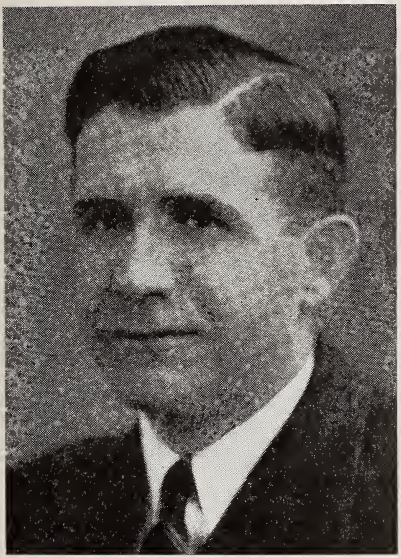
By

R. A. MOYER

Research Associate Professor in Highway Engineering

Iowa State College

Ames, Iowa



When Chairman McKinnon invited me to present a paper on "The Importance of the Design, Construction, and Maintenance of Bituminous Surfaces to National Highway Safety," I had the impression that he intended to make the subject so broad that I could discuss the phases of rural highway safety of which I had made special study and which I believed to be of interest to the engineers attending this conference. Probably I am taking a broader point of view than he intended. Nevertheless, in discussing the important subject of highway safety in a paper for the first time before a group of western highway engineers interested in all phases of rural highway work, I would like to present as briefly as I can my views in regard to the general requirements for rural highway safety, giving special emphasis to the way in which bituminous surfaces can contribute to greater safety. Furthermore, I am confident, that this conference will be interested in knowing about the extensive program of highway research of the Engineering Experiment Station at Iowa State College which was started by Dean T. R. Agg 20 years ago. Much of the work which I have been doing during the past 10 years as a part of this program has a direct bearing on the subject of this paper and I welcome the opportunity to present at this time the most significant results of this work.

The distinctive feature of our highway research program from its inception to the present time as compared to programs of other organizations conducting highway research has been the special emphasis given to the effect of the road on the vehicle. This has involved studies of motor vehicle operating costs, gasoline consumption, tractive resistance, tire wear, and coefficients of friction or skidding characteristics of various types of road surfaces or conditions of these surfaces. Bituminous surfaces have been given their fair share of study and part of the program tentatively planned for the future involves studies of the bituminous surfaces which are rapidly taking a leading place among highway surfaces in every part of the country.

HIGHWAY SAFETY — A NATIONAL PROBLEM

During recent years the mounting accident toll on our highways has been a matter of grave concern to the users and builders of our highways. In every section of the country the picture has been very much the same. In cities and states where a steady reduction in highway accidents has been obtained over a period of years, it is generally conceded that this reduction was made possible only by a well-organized program to reduce accidents in which all divisions of the state and city highway and street departments worked in close cooperation with each other and with the highway patrol, city police, state and local safety councils, motor clubs, and others interested in promoting highway safety. The promotion of highway safety is a definite responsibility of every highway administrator and highway engineer. As I will try to show later, it is an important consideration in practically every phase of highway work including the economics or financing, the design, construction and maintenance of highways, and the control and regulation of traffic.

Accident Statistics and Cost of Accidents

The typical engineering procedure in the study of problems of this type is to examine the facts. All highway engineers, it seems to me, should be interested in information concerning types, causes, and extent of highway accidents. Also, all available information in regard to the probable cost of highway accidents should be considered in developing plans for the expenditure of highway funds. The best source for general information in regard to accidents of all kinds is in ''Accident Facts'' compiled and published annually by the National Safety Council. In the 1939 edition of ''Accident Facts'' 32,400 fatalities and 1,150,000 nonfatal injuries including 90,000 permanent disabilities were the casualties reported on the nation's streets and highways during 1938. The direct costs of these accidents were estimated at $1,500,000,000. The accident rate on rural highways was reported as increasing and in city or urban areas as decreasing. For 1938 approximately 60 per cent of the highway accidents were rural. Since recent traffic surveys in the State Wide Highway Planning Surveys indicate that approximately 75 per cent of all rural highway traffic is on main rural highways, the direct costs of accidents on main rural highways for 1938 may reasonably be assumed to have amounted to $700,000,000. This is 87 per cent of the amount state highway departments received from all gasoline taxes and motor vehicle license fees during 1938. If modernizing our rural highways will reduce rural highway accidents, and I am confident that it will, then it would seem to be much wiser to invest in accident-proof construction a sum equal to a substantal portion of the savings to result from lowered accident costs on modernized highways than to continue to build the less costly but more hazardous types of highway. This means, of course, that increased highway revenues are needed in practically every state. To convince the motoring public that the more costly highways are safer and will in the long run provide cheaper transportation than many of our existing roads, is a problem not only for the highway administrator but one which every highway engineer should take to heart. An annual accident cost of $700,000,000 on our main state highways is too great a price to pay when we know full well that highways can be built with a real factor of safety which will greatly reduce this cost.

The following analysis of rural highway accidents taken from the 1939 edition of "Accident Facts" provides a list of the types of accidents which in my opinion can either be eliminated or greatly reduced on main state highways designed with an adequate margin of safety. The relative importance of the various types of accidents listed as indicated by their frequently seems to me to be particularly significant.

Location and Movement:	Fatal, Per cent	Nonfatal, Per cent
Intersection—collision, two motor vehicles	7	18
Intersection—pedestrian accident	2	½
All Rural Intersection Accidents	10	20
Nonintersection accidents:		
Head-on collision	10	7
Angle or sideswipe	10	17
Left road—struck fixed object—on curve	3	2
Left road—struck fixed object—on straight road	5	4
Left road—then overturned—on curve	8	6
Left road—then overturned—on straight road	12	11
Pedestrian accidents—car going straight	24	5
All nonintersection accidents (rural)	90	80
Total for types of accidents listed above	82	72

It should be noted that in the above analysis no mention is made of the cause of the accidents, as to whether the accidents were caused by a driver failure or some error of judgment on the part of the driver, a vehicle failure, or a road failure, such as defective brakes, skidding on a slippery road, poor visibility, etc. It is evident that of the items listed, which comprise 82 per cent of the fatalities and 72 per cent of the nonfatal injuries, practically all are of a type which can reasonably be expected to be eliminated on a four-lane divided highways with grade separations and road widths, shoulder widths, side slopes, etc., designed with an adequate margin of safety considering the volume and speed of traffic on the given road. Even on a modernized two-lane highway many of the types of accidents listed above can be eliminated or greatly reduced.

A striking fact brought out by this analysis was that only 10 per cent of the fatalities and 20 per cent of the nonfatal injuries occurred at intersections which means that our most important problem has to do with the reduction of rural highway accidents occurring at points along the road between intersections. Equally surprising was the fact that 26 per cent of all rural highway fatalities involved pedestrians. Large sums of money are spent in this country to improve intersections but rarely are special provisions made for pedestrians. In fact, the pedestrian is not even considered in most highway plans. The construction of smooth, hard bituminous footpaths particularly near cities and towns should be a real factor leading to the reduction of pedestrian fatalities on rural highways.

It is significant that more accidents occur on straight roads than on curves and that cars leaving the road and overturning, head-on collisions, and sideswipes are next in the order of their importance. Thus, in modernizing our highways, the design of straight sections of highway should be given as much consideration in the interests of safety as the design of curves. There is evident need of greater road widths, shoulder widths, and flatter side slopes, all of which can readily be embodied in plans for bituminous surfaces at lower cost than for the more expensive types of rigid pavements.

While the above analysis of accidents is very general, it indicates the value of accident statistics in planning highway improvements. For more thorough studies every highway department should maintain a spot map and an accident file with complete descriptions of accidents. The purpose of the spot map is to reveal high accident locations or concentrations. Drivers repeatedly get into trouble at the same places. Deficiencies in the highway can generally be indentified as contributing causes to these accidents. The identification and treatment of these locations is the first job of the engineer from the standpoint of safety.

With the above accident statistics and accident costs in mind, let us now consider how the various factors related to the design, construction, maintenance, and cost of bituminous roads fit into a program for the promotion of safety on the highways of the nation.

Designing Highways for Safety

For many years the chief concern of highway engineers was to lift traffic out of the mud by building all-weather roads or to smooth rough and rocky roads so that motorists could travel at any time of the year at moderate speeds with a fair degree of comfort. To meet the insistent demand for surfaced highways, the engineers tried to make the highway dollar go as far as possible with the result that a large part of our road mileage today has narrow pavements, narrow shoulders, narrow bridges, steep side slopes, sharp curves and curves with inadequate superelevation. I do not mean to criticize these engineers. With limited funds they did what was best under existing conditions and in accordance with accepted standards of their day when vehicles were fewer and speeds lower.

Today, however, it seems to me we have reached a critical period in our highway development and we should examine very carefully the program which should be undertaken during the next decade. New construction is needed in many places to provide additional capacity for increased traffic. However, there is a greater and more immediate need for modernization of existing primary highways to provide greater safety. We can no longer compromise highway dollars and safety to the same extent we have in the past.

As pointed out previously, accident statistics indicate the need for wider pavements and wider shoulders. Also, traffic behavior studies, made as a part of the Michigan highway planning survey, show that the higher the speed the greater is the demand for pavement edge clearance and for clearance when passing other vehicles. Today we find many expensive high type pavements with narrow, soft shoulders and shoulders not built up level with the pavement surface. Main state highways designed with an adequate margin of safety require shoulders sufficiently wide to furnish a parking space for buses and trucks 8 feet wide with an edge clearance of at least 2 feet for the drivers when making emergency repairs. In addition, they should be sufficiently firm and free from obstructions so as to provide an auxiliary traffic lane for use in emergencies. For these reasons the recommended standard width of both traffic lanes and shoulders on main state highways is 12 feet. In the very excellent report by the Bureau of Public Roads made this year to the President and Congress entitled, ''Toll Roads and Free Roads,'' the recommended standard cross sections for two-lane and for four-lane divided interregional highways are shown. The traffic lane widths recommended are 12 feet and shoulder widths 10 feet. I consider shoulder widths of 10 feet on heavily traveled highways as a reversion to the old policy of compromising highway dollars with safety. It is quite obvious that on secondary highways or highways with low traffic volumes, the cost of 12-foot traffic lanes and 12-foot shoulders would be prohibitive and in fact there is not the same need for such widths when the traffic volumes are low. However, on main state highways the design standards should be based on the physical requirements of the moving vehicles with an ample margin of safety and cost should not require that the design standards be modified under any condition. I make no exceptions here even for main state highways through difficult mountain locations. Such locations may require that lower speed standards be adopted with corresponding changes in curvature, but as far as pavement and shoulder widths are concerned, I recommend that the 12 foot widths be retained.

The cross section shown in Fig. 1 illustrates the more important feature of a bituminous road using design standards which provide an adequate margin of safety. It should be noted that the bituminous construction extends over the full width of road from edge to shoulder to edge of shoulder. The center line marking almost amounts to a parting strip being two feet wide and is finished with light colored coarse textured aggregate rolled into the bituminous surface periodically as needed. This two-foot width is not a part of the traffic lanes. It is wide enough to permit the use of special markings which can easily be seen by the driver when no-passing zones are to be designated in this way. A traffic lane marking constructed in the same way as the center-line marking except that its width is only one foot, clearly defines the edge of the roadway. The bituminous construction extends to the edge of the shoulder as a thin mat over a well-compacted subgrade. The integral construction provided in this way is a distinctive feature of the bituminous construction which would be very costly if attempted with the rigd type of pavements.

Bituminous road surfaces have been criticized because the black surface absorbs light at night. Some states, notably Wisconsin, have used light colored surface with good visibility at night and no glare during the day time. Where such a surface finish can be built at a reasonable cost, the center line and traffic lane markings should be made with a contrasting colored paint such as yellow or black and for the same widths as recommended above. The above design may be criticized as providing the equivalent of an undivded four-lane roadway, a type of construction for which high accident rates have been reported. However, these high accident rates occur only when the four-lane roadways are carrying traffic far in excess of the 1,500 to 2,000 vehicles a day which is the maximum average daily traffic recommended for a two-lane roadway designed with an adequate factor of safety. When the average daily traffic volume exceeds 1,500 to 2,000 vehicles, the four-lane divided highway designed to eliminate all conflicting traffic movements is fully justified if an adequate margin of safety is to be provided.

There are three important safety features which characterize the design in Fig. 1: (1) Good visibility and clearly defined traffic lanes during both day and night time with the road wet or dry, (2) easy maneuverability, and (3) high resistance to skidding when wet or dry.

Good visibility implies not only that light conditions must be right on the road surface but also that the roadway and right-of-way widths must be great enough to make safe driving easy for the motorists.

Easy maneuverability is obtained by using a wide roadway and shoulders and flat side slopes which provide the space and type of surface on which the driver can easily maneuver his car out of difficulties whenever they arise. Until recently vertical curves were designed using sight distances no greater than were considered necessary to stop both cars should one of the drivers be caught unexpectedly when attempting to pass another car. We now know that drivers will not stop but attempt to pass, thereby causing many head-on collisions and sideswipes. While highways can be designed using vertical curves with the 2,000 or more feet of sight distance required for clear visibility when passing, the lengths of the curves and the grading costs involved would be so great in many cases as to make such construction impractical. The better plan seems to be to provide ample road and shoulder widths, and flat side slopes which will make it easy for the driver to maneuver his car out of the difficulties which may arise in the passing operation

The surface requirements to provide high resistance to skidding will be discussed in detail in the section of this paper devoted to construction features.

Important design features in which safety is a primary consideration tion but which are not directly related to the road surface are the design standards related to speed, curvature, supelevation, widening on curves, and the use of transition curves or spirals. Road tests have demonstrated that bituminous surfaces of the type shown in Fig. 1 constructed with an abrasive nonskid surface finish, will provide a greater margin of safety than is commonly obtained on the rigid type of pavements of today. This extra margin of safety has money value which should be credited to the bituminous surface. I consider the design standards related to speed, curvature, superelevation, widening, and transition curves as of equal importance for the promotion of safety as the standards for the cross section and the type of surface construction. Time will not permit going into detail in regard to these design features. I would, however, like to emphasize the need for, and the method I advocate for, providing a uniform factor of safety so that the motorist may expect the same margin of safety at all points along a given section of highway designed for a predetermined uniform speed.

Several years ago it was reported in the Engineering News-Record that in several West Coast states highways were "designed" for a speed of 100 miles per hour, presumably with the purpose in mind that highways "designed" for a speed of 100 miles per hour would provide a substantial margin of safety for the present traffic with speeds ranging from 30 to 70 miles per hour and if per chance speeds of 100 miles per hour were to develop in the future, the highways in question presumably would be safe for such speeds. While the end result may be the same, I believe sound engineering practice would require that the given major state highway be designed for a speed of 60 or 70 miles per hour with a factor of safety of three or more, thereby making a speed of 100 miles per hour permissible but obviously not necessarily safe.

Briefly, I would recommend design speeds of 70 miles per hour for important inter-regional highways (also recommended by the Bureau of Public Roads in "Toll Roads and Free Roads"), 60 miles per hour for main state highways, and 40 or 50 miles per hour on difficult mountain locations and on secondary roads depending on the volume and character of traffic served. In determining the limiting radius of curvature for each design speed on the various road systems, I would use the standard curve formula, $R = 0.067V^2/(e+f)$, which was definitely established as a sound basis for curve design in our road tests on curves during 1932 to 1934. In this formula, R is the radius of the curve in feet, V is the speed in miles per hour, e is the superelevation in feet per foot, and f is the coefficient of friction to resist skidding sideways. In localities where ice is likely to form on the road, the maximum safe value for superelevation is 0.10 ft. per ft. Road tests and observations of racing drivers have demonstrated that the maximum possible coefficients of friction which can be developed on curves range from 0.5 to 0.6. If a factor of safety of 3 or 4 is provided, a coefficient of 0.15 is obtained and this is the value recommended for design purposes. It should be noted that cars operated at the design speed depend more on friction ($f = 0.15$) than on superelevation ($e = 0.10$) to hold the car on the curve. However, the distribution of speeds of cars on curves is generally such that 80 to 90 per cent of the traffic is traveling slower than the design speed, and in fact, on rural highways there are many vehicles operated at or near the average speed which require little or no friction to hold them on the curve. It will be seen then that the design of curves at best is a compromise due to the large variations in traffic speeds. However, in the interests of greater safety, curves should be desgined with a factor of safety of 3 or 4 for the maximum uniform speed of the faster group of drivers.

Our road tests on curves also demonstrated that spiral transition curves are necessary to drive cars safely at the high speeds of today and especially at the higher speeds which are almost certain to develop in the future. The formula for the length of spiral (Ls) established in our road tests as a sound basis of design and now widely used as a standard is $Ls = 3.16V^3/CR$ in which C is the critical term in the formula and represents the rate of change of acceleration in drivng from the tangent to the circular curve as reflected by the change in the position of the steering wheel in completing this operation. Skillful racing drivers have been observed to make this change in the position of the car and the steering wheel at rates corresponding to values of C = 6 ft./sec.^{3} If a factor of safety of 3 is applied to this value of C, a design value of C = 2 ft./sec.^{3} is obtained. Road tests have demonstrated that this value of C is entirely safe and reasonable for design purposes and as pointed out in the curve formula, if this is the value used at the design speed, then the large group of drivers driving at or near the average speed, will develop a value of C of about 1 ft./sec.^{3}, the value which Royal-Dawson, the British authority on this subject, contends is the only proper value for design purposes.

On the basis of the rational and experimental approach to the technical highway problems outlined above, the most radical changes in our highway development appear definitely to be in the design field. Safety is the prime consideration in this program. In view of their low construction cost, the ease of their construction, and the excellent nonskid surface which they can provide, bituminous surfaces are bound to play an important part in the future highway development as it relates to traffic safety.

The Construction of Bituminous Surfaces for Safety

Elements in the construction of bituminous road surfaces which contribute to safety are (1) the nonskid property of the surface, (2) the property of absorbing light or of reflecting light (glare), (3) the smoothness or roughness of the surface, and (4) the elimination of the dust hazard found on untreated gravel and macadam surfaces.

Skid Tests and Nonskid Surfaces. Since 1932, an intensive study of many phases of the problems involved in skidding and of constructing nonskid surfaces has been conducted under my direction at Iowa State College. Road tests have been run on many surfaces and surface conditions, to determine the extent of the slippery condition, the probable causes, and the most satisfactory methods for correcting this condition. The frictional requirements when braking and when driving on curves have been studied. The variable effects due to the conditions of the tires, the load, the type of wheel suspension, and the speed of the car have also been studied. Interest on the part of engineers in developing non-skid road surfaces has been increasing steadily during recent years because (1) many serious accidents have been reported as definitely due to slippery road surfaces, (2) the demand for operating in safety at high speeds has stimulated the construction of nonskid surfaces, and (3) the development of many new types of surface, particularly of the bituminous types known to be slippery if not properly constructed, has also added to this interest. Nevertheless, since the majority of highway accidents are charged to the carelessness of the driver and so few to defects of the road, the real value of nonskid roads is not always appreciated by the public or even by all highway engineers. Just because cars do not always skid by sliding end for end or broadside over the road is no reason why such roads may not be considered slippery. A nonskid surface will not only reduce the tendency to skid in this extreme way but what is probably more important, but less evident, it will permit the driver to stop the car in far shorter stopping distances, to make sharper turns, and to maneauver the car with a far greater margin of safety than on a slippery surface.

Statistics collected by the National Safety Council in 27 states revealed that 5 per cent of all accidents were directly attributed to skidding The most significant statistical analysis of skidding accidents was recently reported for the state of Ohio by the National Safety Council Committee on Winter Driving Hazards of which I happen to be chairman. This analysis covers rural accidents only and is as follows: 80 out of 6,242 or 1.3 per cent of dry pavement accidents were skidding; 481 out of 1,747, or 27.5 per cent of wet pavement accidents were skidding; 521 out of 1,045, or 49.9 per cent of the accidents on snow and ice were skidding. It is evident from this report that dry pavements present no serious skidding hazards but that as far as the pavement surface is concerned, wet pavements can be and frequently are slippery and are the cause of many accidents.

Our tests of more than 25 different types of surfaces have provided convincing evidence that roads which are slippery when wet exist because of improper construction and inadequate maintenance. It is possible to build bituminous surfaces that are as free from the dangers of skidding as any road surface in common use today, and with the information now at hand there is no reason why every mile of bituminous road cannot be made safe against the hazard of skidding.

An examination of the bituminous surfaces which the tests indicated as offering high resistance to skidding, revealed that they were covered with sharp, hard sand, or with hard and finely divided crushed rock particles, held in place by the weaker bituminous cement. Since the tops of the sand or rock particles were uncoated with asphalt, these surfaces were reported to have a sandpaper-like finish, which as indicated by the age of the pavement was renewable under the action of traffic and weathering. An examination of the bituminous surfaces found to be slippery revealed that they were covered with heavy seal coats of excess bituminous material or in some cases the surface was covered with a light glaze of asphaltic material which apparently was a characteristic of that type of asphalt. It should be evident that excess asphalt in a bituminous mixture is not only certain to cause a slippery surface in hot and wet weather but is also likely to cause the surface to creep or shove or to form ruts under the action of traffic.

Certain emulsions or cutbacks of the medium or rapid curing type seem to coat the aggregate particles in thinner and stronger films than do straight oil asphalts or asphalt cements. These thin films wear off faster, especially on hard, nonabsorptive aggregate such as silica sand, disintegrated granite, quartzite, etc., than the thicker films obtained in the use of the straight oil asphalts or asphalt cements. For this reason emulsions and cut-backs can be expected to develop a higher skid-resistance with ordinary construction methods than the oil asphalts or asphalt cements.

On various trips which I have made during the past two years, covering about 6,000 miles of bituminous surfaces in both the Eastern and Western States, I have observed that the application of a plain seal coat was the most common cause of the slippery condition observed on about 1,000 miles of road and that the surfaces which appeared to be nonskid were constructed without seal coats or with seal coats liberaly covered with crushed rock chips, or clean screenings, or sand and gravel such as the Platte River gravel. The use of a dense graded mix or of fine graded aggregate for cover material applied in a surface treatment of at least two applications seemed to provide the water tightness which is necessary in regions where freezing and thawing may cause disintegration of surfaces not so protected. Where seal coats are intended to be used to provide a watertight, nonskid, wear-resistant surface over a dense-graded mat, I recommend the application of 0.2 gallon per sq. yd. of a cut-back asphalt or a quick-setting emulsion, followed immediately by spreading 35 lb. per sq. yd. of aggregate ⅜ inch to 40 mesh size with a mechanical spreader. The aggregate should be brushed evenly over the surface and then be rolled with a three-wheel, 10 ton roller or a Roll-A-Plane type roller. In the city of St. Louis, 1,800,000 sq. yd. of anti-skid surfacing has been placed using 95 per cent Joplin chats, ⅜ inch to 40 mesh size and 5 per cent asphalt cement (85 to 100 penetration) in a carefully controlled hot plant mix job. No seal coat was used. A large part of this work was done at intersections and at each approach 150 feet back from the intersection. The Director of Streets, Mr. McDevitt, reported that accidents at intersections so treated have been reduced approximately 50 per cent and that the cost of the treatment was surprisingly low. On slippery sheet asphalt pavements 60 lb. per sq. yd. of the nonskid surfacing was used at a cost of about 25 cents per sq. yd. On granite block and brick streets, 135 lb. per sq. yd. was used at a cost of about 50 cents per sq. yd. Mr. McDevitt reported that he received many letters from bus and taxi drivers and others testifying to the effective anti-skid, nonglare qualities of this type of surface.

Work of the type reported by Mr. McDevitt clearly demonstrates the important place which the nonskid bituminous surface construction can occupy in promoting safety on the nations streets and highways.

Light Absorbing and Reflection or Glace Properties. Mr. R. H. Baldock in his report for the United States to the Eighth International Road Congress held at The Hague in 1938 and published in Public Roads, August, 1938, presented some significant conclusions on the subject of light absorption and glare as follows:

1. Dry, light-colored pavements are from 2 to 4 times as luminous as dry, dark-colored pavements. No pavement has much luminosity when wet.

2. The water film present on the surface of a wet pavement acts as a mirror to reflect light and causes dangerous glare.

3. Pavements having a fine, sandpaper texture are covered with a film of water when wet and this film is a very effective reflecting surface (causing dangerous glare). Coarse-textured surfaces having pits or indenta-

tions of from ⅛ to ¼ inch in depth break up the water film and therefore reflect light in considerably less degree.

4. For a maximum of skid resistance and a minimum of glare, it is necessary to construct pavements having surface indentations or grooves deep enough to prevent the surface tension of water from holding a film of water on the surface. Moreover, the small particles of rock or sand should stand above and screen off the light from the water running in the indentations or grooves above described. The surface texture of the pavement should be as fine as possible to accomplish the above objective inasmuch as a coarse, knobby texture reduces skid resistance.

From the above conclusions by Mr. Baldock and as mentioned earlier in this report, it is evident that bituminous surfaces can be constructed with low reflecting or glare properties for both day and night driving, thereby eliminating much of the blinding and fatiguing effect which is caused by glare and which, no doubt, is the cause of many accidents. While it is true that many bituminous surfaces are dark colored and absorb considerable light from the headlights or artificial lights when driving at night, it is possible to construct light-colored bituminous surfaces by a liberal application of light-colored cover material on the seal coat and rolling the light material thoroughly with a heavy roller to imbed it so that it will not be displaced readily. Seeing at night is poor today even with the best headlights and the highway engineer should assist in every way he can to improve light conditions and to eliminate glare. Until such time as the manufacturers can equip cars with the greatly improved headlights which have been developed by General Electric and Westinghouse, the maximum safe road speeds for night driving will continue to be about 45 or 50 miles per hour. On dark-colored surfaces without wide luminous or light-colored center line or traffic lane markings, the safe speed should be reduced 5 to 10 miles per hour below the above maximum.

Surface Roughness. I am sure that every engineer attending this conference has had the experience of losing steering control of his car temporarily on a washboarded gravel road or a wavy bituminous road especially on a sharp curve on a down hill grade. You may not have had an accident but that was only a matter of your good fortune due to the fact that another car did not happen to be passing you at the time. The ordinary waviness which may be observed on some bituminous surfaces is great enough to cause the tires to bounce clear of the road. The efficiency of the braking and steering operations on such surfaces is greatly impaired. Although I have made no measurements to prove my point, I am confident from observations of properly constructed bituminous roads and from the riding qualities of these roads that bituminous surfaces can be built with a smoothness equal to that of any other type of surface. There is probably no other property of a road surface of which the motoring public is more critical. If you give the motorist a wide smooth surface which is not slippery when wet and which is free from the annoying thumping due to joints or rough spots, he will be a booster for such a road. Give him a road, however, with pot-holes, wavy spots, and a few frost boils and you will have a lot of criticism to face.

During the past year in cooperation with the Bureau of Public Roads, we have been engaged in an exhaustive study of the differences in motor vehicle operating costs and operating characteristics of untreated gravel and of portland cement concrete roads in central Iowa. This has involved operating four test cars daily, two exclusively on a gravel test route, and two on a concrete route, both of which are about 235 miles long. In connection with these tests, measurements have been made of the surface roughness of the gravel and concrete routes using a special recording profilometer device which we developed and using also the Firestone Rubber Company riding comfort meter which consists of multiple-element accelerometers which record the vertical accelerations over a range of from 5 to 500 feet per second per second when placed on the flood boards in the center of one of the test cars. The most significant result disclosed from the road roughness tests conducted to date is that the smoothest mile section on the 235 miles of gravel test route was rougher than the roughest mile section on the 235 miles of concrete test route. On the roughest washboarded gravel, acceleration of 50 to 60 ft. per second per second were measured, whereas on the concrete the accelerations rarely exceded 10 ft. per second per second. If roads are to be built to permit safe operation at speeds of 70 and 80 miles per hour, then specifications which require that no variations greater than ⅛ inch in 10 feet be permitted, should be written into every contract for bituminous surface construction.

Elimination of Dust Hazard and Economic Advantage of Bituminous Construction

There are in this country hundreds of thousands of miles of untreated gravel and macadam roads which are dusty during a considerable portion of the year. These roads are not only hazardous but are very unpleasant and uneconomical to drive on, especially in heavy traffic. In our tests during the past year, involving about 60,000 miles of travel on the gravel roads, we have had 40 punctures. During the same time and in the same distance we have had only 1 puncture on the concrete roads. The total tire costs on gravel or macadam may be double the tire costs on a smooth hard surface. Gasoline costs are about 15 per cent higher and oil costs are also higher due to the effect of the dust and surface roughness. Maintenance costs for fender-pitting, broken windshields and light lenses, etc., will also be greater on gravel than on concrete. Our studies of mail carrier car costs and the present road tests indicate that a difference of about 1 cent per mile in operating cost can be accounted for when comparing car costs on gravel and concrete. While data are not yet available for an accurate comparison of bituminous surfaces there is indication that the difference in operating costs between gravel and bituminous surfaces may be nearly the same as the difference between gravel and concrete

With an average daily traffic of only 100 vehicles a day and an average saving in operating cost of 1 cent per vehicle mile, the extra operating costs would amount to $365 per year and would justify an annual expenditure of $365 for a bituminous surface treatment of a gravel or macadam road if this expenditure is considered on a maintenance basis only. With a traffic of 500 vehicles daily, and there are many miles of dusty gravel and macadam roads in this country which carry this traffic volume, the extra operation cost would be $1,825 annually. This certainly seems to be too great a penalty to pay when it is known that a bituminous surface can be provided for this traffic volume for less than half this extra cost on an annual basis, and which has the advantage of greater safety, driving comfort, time saving, and lower operating cost than are available on the dusty and rough gravel and macadam roads.

Maintaining Highway Surfaces for Safety. There is no question that the best policy for maintaining highway surfaces is to build the maintenance into the road rather than to be continually tacking on to or rebuilding the road as a maintenance operation. Traffic will be benefited in many ways if this policy is followed and greater safety will be one of the important benefits. Pavement widening and shoulder repair work are the most common maintenance operations related to safety which should be anticipated when the road is constructed.

In routine maintenance work, no other materials are used so universally as the bituminous surfacing materials which at the same time have so many possibilities for promoting safety. The greatest value in this respect is when used as a nonskid resurfacing or patching material on rough and slippery concrete, brick, stone block, and asphalt surfaces. Low and rough spots may be made smooth, light and glare qualities may be improved and other surface defects may be corrected.

Bituminous surfaces which are glazed over with excess asphalt and which are known to be slippery may be improved by scarifying the surface, by applying screenings or rock chips uniformly over the surface heated either by the sun or by a heater, honing or planing and mixing it with a blade, and finally rolling the surface with a heavy roller. Another possibility is that of cutting back the excess asphalt with a volatile solvent, such as kerosene or a fuel-oil distillate, then applying gritty cover material to blot up the excess binder, mixing it with the blade or hone, and then rolling it. While limestone dust and powdered asphalt are very effective in drying an oily surface or in blotting up the excess asphalt and in strengthening or stabilizing the surface, they lack the grittiness which sand and coarser screenings and rock chips can give the surface to make it highly skid-resistant. The limestone screenings or rock dust or powdered asphalt should only be used in combination with the abrasive aggregate required to make the surface non-skid. The use of large stones will develop a surface with a rough-knobby finish which our tests have shown to be less skid-resistant than surfaces with a sandpaper finish or the gritty chipper finish using a hard ⅜ inch to 40 mesh size aggregate. The knobby finish also creates a rumbling noise when driving over it which is annoying and uncomfortable to passengers in the car.

SUMMARY

In this paper I have tried to show the important part which bituminous surfaces can play in a reduction of the $700,000,000 annual accident cost on the main rural highways of our country. From every point of view bituminous surfaces were shown to be well suited to fit into the important program of modernizing the highways of this country which the State Wide Highway Planning Surveys and a strict engineering analysis of the design and construction problems have shown are urgently needed.

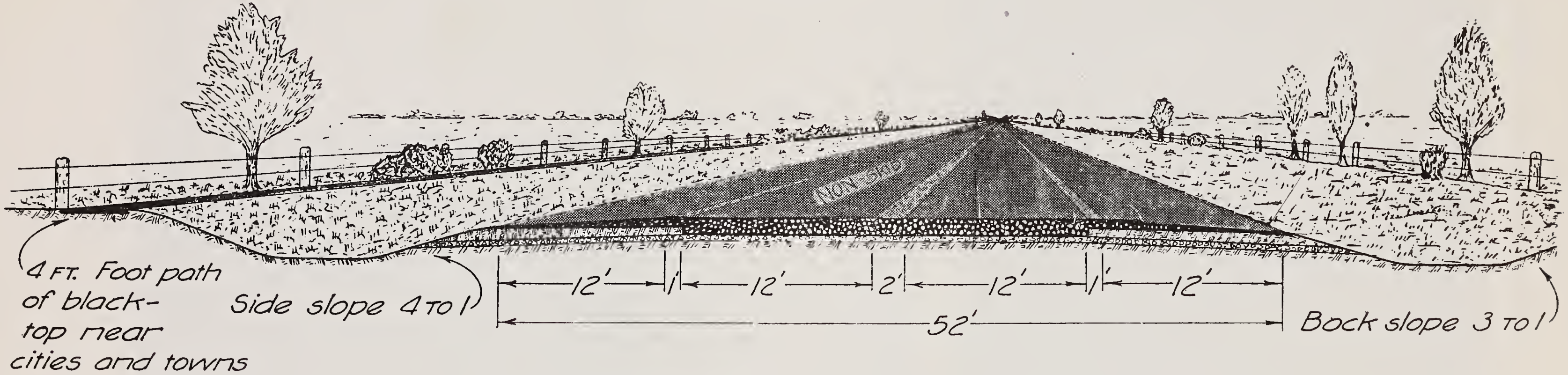
We should no longer compromise highway dollars with safety as we have in the past. Highways and highway surfaces can and should be constructed with an adequate factor of safety just as buildings and bridges are designed and constructed with a factor of safety. Safe highways will cost more now but in the long run the total transportation cost will be less. To obtain the increased funds necessary for financing this program of construction, we must sell the public on the value of safe highways since if new funds are not made available, a balanced program of design and construction will not be possible or at least will be greatly delayed.

Accident statistics for rural highways indicate the need for pedestrian foot paths near cities and towns which can be built of bituminous materials at low cost. Also, with more accidents on straight roads than on curves, there is need for as much regard for safe design standards for the highway cross-section as for the design standards on curves. There is need for greater pavement and shoulder widths, for smooth non-skid surfaces free from glare and with good light properties at night, all of which can be provided with bituminous surfaces at a moderate cost. There is, also, need for spiral transition curves and circular curves designed with proper superelevation to make the ride comfortable and steering easy and with a factor of safety of 3 at the design speed against skidding sideways to prevent the car from sliding off the curve. This factor of safety can be assured if a satisfactory nonskid surface is provided such as is possible with the best non-skid bituminous construction.

Today, we know that bituminous surfaces can be built to meet variable traffic demands which will provide cheap and safe transportation on our highways. We know that slippery surfaces exist because of improper construction and maintenance. In most cases the methods for building and maintaining non-skid surfaces are so simple that no special technical knowledge is necessary to produce good results. With the present knowledge of design, construction, and maintenance, and if funds are made available to apply this knowledge in a workable program of modernization in which bituminous surfaces will play a prominent part, I am confident that accidents on this nation's streets and highways will greatly be reduced.

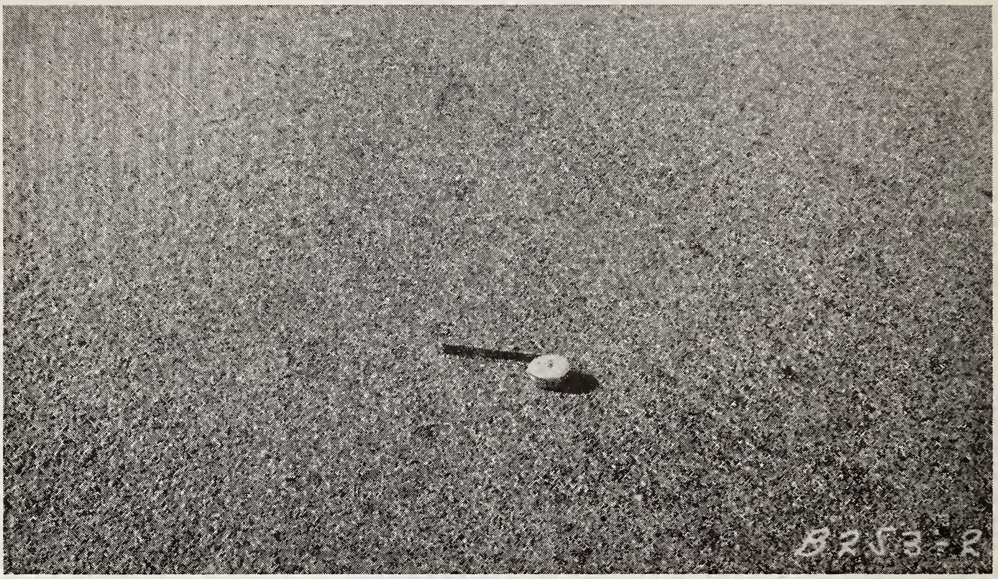
FIGURE 1.

ELEMENTS OF A TWO LANE BITUMINOUS PAVEMENT
DESIGNED TO PROVIDE AN ADEQUATE MARGIN OF SAFETY
Good Visibility — Easy Maneuverability — High Resistance to skidding
12-FOOT TRAFFIC LANES AND 12-FOOT SHOULDERS


NON-SKID
12'
1'
12'
2'
12'
1'
12'
52'
4 FT. Foot path of black-top near cities and towns
Side slope 4 TO 1
Bock slope 3 TO 1

TRAFFIC LANE MARKING
Center-line 2 feet wide — Side-lines 1 foot wide
(Made of 1 inch light-colored crushed rock or gravel rolled
into black-top surface.)

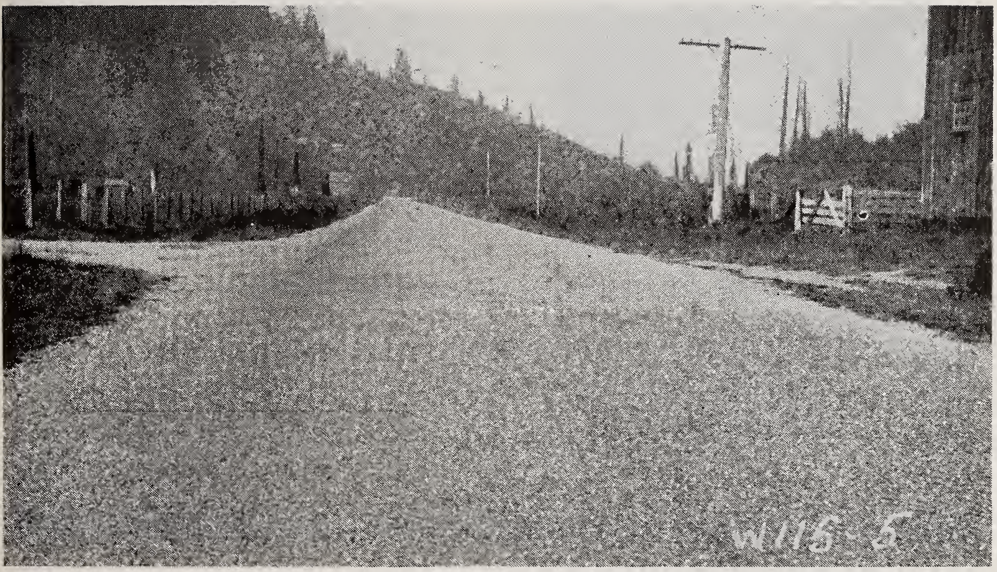
NON-SKID SURFACE ON ASPHALT CONCRETE



Non-skid surface on asphalt concrete as developed by the California Highway Department using cut-back asphalt or emulsified asphalt seal covered with ⅜″ to 30 mesh screenings to develop gritty, sandpaper finish.

NON-SKID SURFACES ON OILED GRAVEL ROADS

Hot oil (90-95 road oil) seal (fine screenings).
This is "sandpaper" type finish and is preferable to use of coarse screenings.



Hot oil (90-95 road oil) using coarse screening for non-skid bituminous macadam. This rough textured, open-graded type of surface finish provides less skid resistance than when fine screenings are used.

Highways of Tomorrow

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OBSOLESCENCE

There is a large mileage of highways still in use which were built more than twenty-five years ago. At that time 20 miles per hour was considered a reasonable speed for vehicles and 30 miles per hour was considered a top limit. There was no intimation at that time that speed limits would double and for that reason the engineers can hardly be blamed for the present defects in highway design.

More than 14 per cent or about 11,000 miles of the present surfaced state primary system was built prior to 1921 and has not changed to any extent. This mileage is entirely inadequate for the high speed traffic of today.

The speed of motor vehicle traffic increased to some extent during the 1920-30 period and it was during this period that much of the existing mileage was built. This mileage was designed for higher speeds than that built during the preceding decade but is a long way from being adequate for present traffic.

There was a sudden increase in motor vehicle speeds during the early thirties and this increase coupled with the more recent uptrend in traffic volume has created an urgent demand for a program of modernization to include straightening, improving grades and widening.

LACK OF BALANCED PROGRAM

There has never been a uniformly balanced program of highway improvement in the various states. The rapid growth in the number of automobiles and the rather sudden increase in vehicle speed during the past decade has resulted in an unbalanced improvement program among the several highway classifications throughout the country.

This unprecedented growth in motor vehicle registration made it impossible to foresee the future highway transportation requirements and resulted in rendering a large mileage obsolete in a comparatively short space of time.

The recent national traffic survey just completed has resulted in the collection of the kind of information necessary to formulate a practical program of highway improvement where it is most needed and for the first time makes possible a balanced construction program between primary highways, secondary and feeder roads, and city streets.

These three systems form the nation's highway transportation network. It is estimated that 11 per cent of the total mileage carries 57 per cent of all the traffic. These are the primary highways which includes the state and federal-aid system. Streets in municipalities, comprising 6 per cent of the total mileage carries 30 per cent of the traffic and the secondary system which has 83 per cent of the mileage carries 13 per cent of the traffic.

CURRENT STATUS

There are 3,033,713 miles of highways in the present rural system. Of this mileage the state highway departments have jurisdiction over 465,237 miles, 177,000 miles of which may be termed secondary and 288,237 primary. There are 223,000 miles of federal-aid highways included in the primary system.

410,628 miles of the state and federal system had been improved up to January 1, 1939 and of this mileage there was about 130,000 miles of hard surfaced pavement. The remainder is of the lower cost types.

There are 2,568,476 miles of highways under the jurisdiction of the counties and townships. These are mainly farm-to-market roads and secondary roads which are feeders into the primary system. There are no accurate records concerning the status of improvement of this mileage but it is probable that more than 700,000 miles of this system has had some improvement, with a comparatively small percentage of this hard surfacing.

It is estimated that there is about 100,000 miles or about one-third of the primary system which needs reconstruction. This mileage no longer meets the demands of modern vehicle operation due to poor location, narrow roadway and bridges, and slippery worn out surfacing. This mileage is responsible for a large part of the highway death toll.

TRAFFIC DEMANDS

Planning surveys indicate the existing character of automotive traffic including the volume and the origin and destination. It has been found that traffic does not move long distances from its source and the points of concentrated traffic are the centers of population. Therefore the necessity for a system of multiple lane superhighways running from coast to coast and from north to south does not exist at the present time. The maximum extent of local use out of population centers is 30 miles and this can be scaled down to 10 miles around the small cities.

Thirty miles from the heart of a city is the maximum distance of local influence. Obviously there is a certain amount of ''through'' long distance traffic which travels beyond this 30-mile zone but the amounts are small as compared with the local traffic operating within the confines of the area. For instance in the Cleveland area, only 1½ per cent of the traffic on the main roads crosses the area between outside points. On the other hand, over 95 per cent of the daily passenger car traffic entering or leaving the city of Cleveland does not travel beyond the limits of the regional area, 30 miles from Cleveland.

NEEDED HIGHWAY PROGRAM

The greatest problem concerning vehicle movement at the present time is that of getting traffic in and out of cities. There is needed divided four-lane roads separated from cross traffic, also belt lines to accommodate through traffic, and distribute it to other routes. These are particularly necessary in the northeast where the centers of population are close together.

The national traffic survey just completed has resulted in the selection of an inter-regional highway system which utilizes to a large extent existing routes. This system of 27,000 miles of primary highways has been chosen for thorough modernization. It will accommodate 12.5 per cent of the highway traffic on less than one per cent of the road mileage and is located according to the demands of national defense. On the heavily traveled portions of this system there will have to be limitation of entry points and many of the smaller communities will have to be by-passed.

Continued improvement of the remainder of the primary highway system is imperative with special attention to elimination of hazards such as grade crossings.

There should be continued improvement of secondary and feeder roads with special attention paid to land utilization. Roads should go where the land promises to support the population. It is estimated that there is needed improvement of 700,000 miles of farm-to-market roads not on the state system which would bring relief to about 4,000,000 farmers now living on mud roads.

The selection of these feeder roads to best serve agriculture is an important step in the development of farm lands. As a result of recent federal legislation by Congress appropriating funds for secondary roads, the state authorities in cooperation with the counties will select a system of farm-to-market roads, not exceeding 10 per cent in each state, for improvement.

The state-wide planning surveys now under way should give the information necessary to select the roads of high relative usefulness and should lead to the abandonment of areas lacking fertility and start a movement toward more profitable farming areas.

In a well balanced program of highway improvement there must necessarily be complete information on all economic phases. The federal government, the state and the county are the jurisdictions responsible for this work. The first named jurisdiction, heretofore principally concerned with the federal-aid system of primary highways is now participating in secondary road construction in cooperation with the states and counties. This should result in coordinating farm-to-market roads at it has the state highway systems.

FINANCING

There has been little uniformity in the financing of highways between the several jurisdictions, state, county and city. The largest source of state highway income, highway user funds, are shared with the counties in all but six states, and 28 states use a certain percentage of these funds for purposes other than highways. The changing picture in highway financing is illustrated in the following statistics:

	1923	1928	1933	1937
Bonds	19 %	14.2%	3.6%	5.3%
General Taxes	13	6.3	3.6	0.3
Miscellaneous	3.3	1.5	2.0	0.4
Motor Vehicles Fees	31.5	30.5	23.2	24.0
Gasoline Taxes	3.4	27.5	42.0	45.2
Federal Aid	15.4	10.0	24.2	23.4
Local Transfers	14.4	10.0	1.4	1.4
	100.0	100.0	100.0	100.0

There is need for a definite annual federal-aid policy for stabilized financing. While there has been only one lapse in federal participation since 1916 which lasted through a two-year period, there is no law that makes this participation mandatory. This is not only true of the regular federal-aid, but also federal-aid grade crossing funds and secondary road funds.

The other necessity for stabilizing the highway program is the assurance in each state that highway user funds go into highways. There are seven states which have constitutional amendments against the use of motor vehicle taxes for non-highway purposes, several more of them do not divert funds, but the bigger percentage of them use part of this income for other purposes. The principles of equitable taxation require these funds to be used for highways.

REQUIREMENTS OF A BALANCED PROGRAM

A balanced program of highway development will include the construction of a system of direct interregional highways with the necessary connections around and through cities, these highways to conform to the War Department's plan for defense and long range peacetime traffic.

Second, the modernization of the Federal-aid system; third, the elimination of hazards at railroad grade crossings and fourth, the improvement of secondary and feeder roads properly integrated with land use programs.

Unfortunately, much of the currently needed modernization program is located in built-up areas and difficulty will be encountered in obtaining rights-of-way for widening, straightening and grade separation work. In fact this problem is greatly retarding the current construction program. It will be necessary, therefore, for the organization of a federal land agency empowered to acquire the land necessary to carry on the program. A right-of-way width of not less than 300 feet is generally recommended for rural areas and a minimum of 160 feet in urban areas.

HIGHWAY DESIGN

Highway alignment must be designed to keep the vehicle operator in the proper lane. The use of transition curves rather than a series of tangents, connected by simple curves is a modern engineering development and increases safety by making driving easier, and relieving the monotony by giving a more pleasing appearance.

In high speed highways, single vehicle lanes should be built to a 12 foot width. On two lane highways sight distances should be from 1200 to 1500 feet. A two lane highway shows congestion at 2000 vehicles a day and when this amount of traffic exists the highway should be widened to four lanes with a dividing strip.

Roadway surfaces must be smooth and skid resistant, should be properly crowned and curves of more than one degree superelevated. Subgrade soil surveys are carried on by most of the state highway departments at the present time and this is resulting in the scientific selection of surface types. This work is in its infancy and it is reasonable to believe that by the result of this scientific soil analysis the bearing power of subgrades will continually be increased.

Modernization factors in highway design can be summarized in general as follows:

1. Elimination of sharp curves and dangerous grades.
2. Superelevation and widening of curves.
3. Increasing lane widths to meet the demands of modern traffic speeds.
4. Elimination of grade crossings.
5. Replacement of narrow bridges with a travelway at least as wide as the roadway.

HIGHWAY IMPROVEMENT PLAN

The National traffic survey classifies the highway improvement plan as follows: Urban areas require the construction of trans-city connections and express highways, also belt-lines and by-passes.

The primary system should have a program of construction to include direct interregional routes between centers of population and the modernization of existing rural highways.

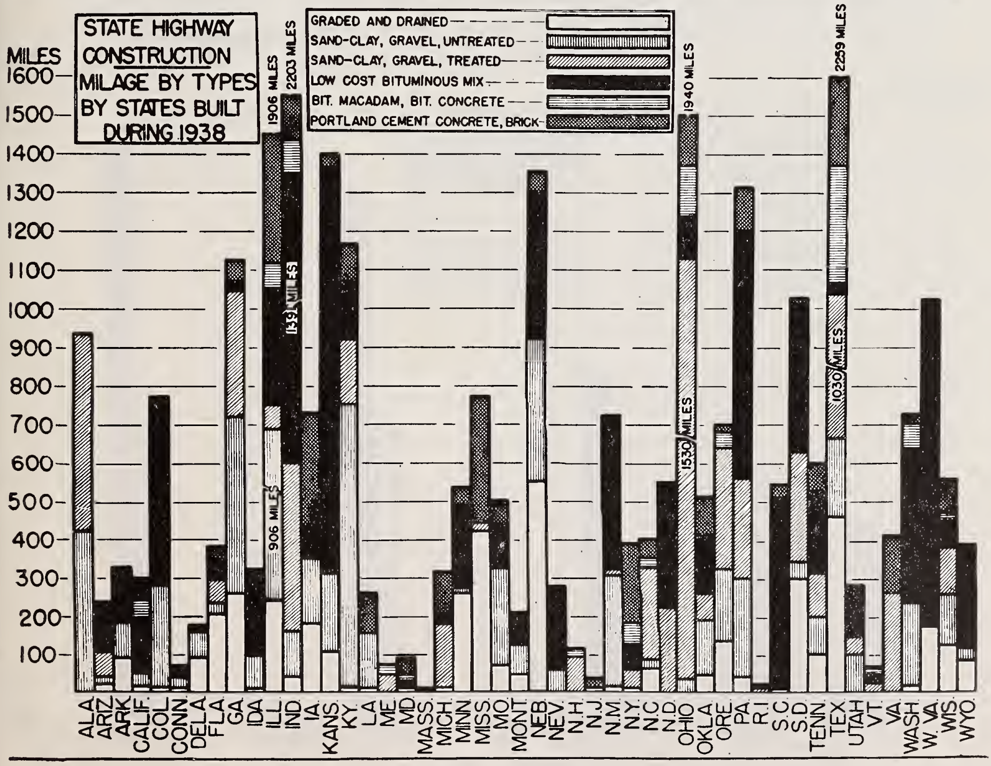
There should be a continuous improvement of secondary and feeder roads.

Adequate financing for a comprehensive program as above outlined will depend on non-diversion of highway user funds and continuous federal-aid program participating in the development of each highway classification. Toll financing of highways is quite definitely out of the picture as a means of obtaining funds for the National system of highways.

THE HIGHWAY OF TOMORROW

Unquestionably, vehicle speeds will continue to increase as well as the size and weights of trucks and buses. Several states are designing highways for speeds of 100 miles an hour. In order to accommodate the vehicular traffic of the future, the high speed or superhighway will include the following features:

1. Total exclusion of pedestrians, cycle and animal traffic.
2. Complete separation of opposing traffic streams.
3. The elimination of crossings at grade and construction of cloverleaf intersections.
4. The separation of high and low speed traffic.
5. Limited entrances and exits, with accelerating and decelerating lanes.
6. Long sight distances, flat grades and easy curves correctly superelevated.



SURFACING

Obviously every type of surface construction will be utilized in a balanced highway program. The types constructed by the state highway department during 1938 may be taken as a criterion. The total mileage under state control is 465,237, of which 177,000 miles are secondary and 288,237 are primary.

7. Full consideration of esthetic effects to make highways fit into the landscape with modern roadside development, wide shoulders and proper drainage.

MILITARY AND COMMERCIAL HIGHWAYS

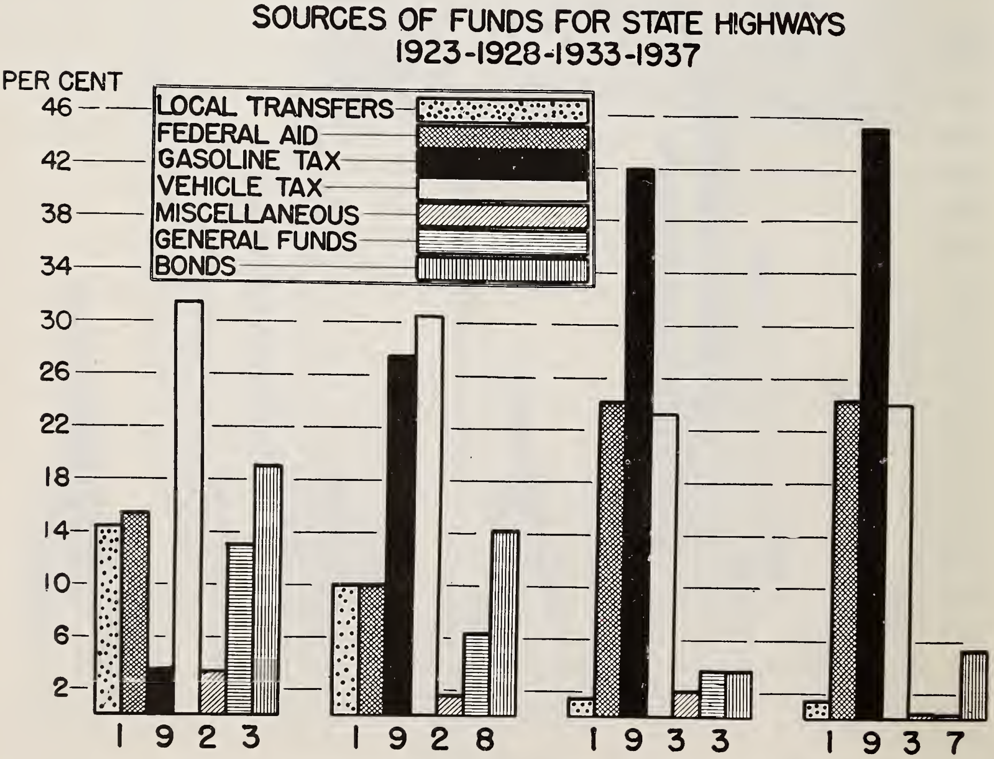
A good commercial highway transportation system also fulfills the transportation demands for military defense. The war department has recommended the construction of certain highways for the use of the military forces in time of war and these routes are in most cases identical with the interregional highways recommended for improvement by the national traffic survey.

This fact assumes importance at the present time as the possibilities of another conflict becomes imminent. By improving the military system the commercial network is extended.

Railroads are easily incapacitated in wartime and the highways must be adequate to carry most of the movement of men and war material.

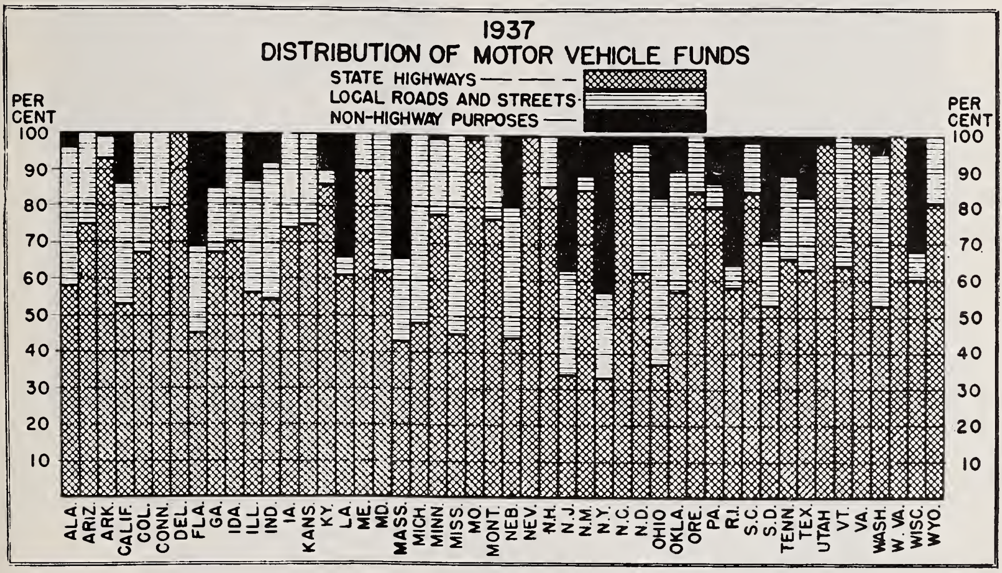
Recent testimony by members of the war department pointed out that movement was essential in war and must take place on roads. Strategical and tactical plans are limited by roads and freedom of maneuvering depends upon their number, quality, location and direction.

Two streams of traffic flow parallel, steadily, and opposite in direction in war, one of full vehicles and fresh men flowing toward the front,



THE CHANGING PICTURE IN HIGHWAY FINANCING the other of empty vehicles and the wreckage of war flowing toward the rear.

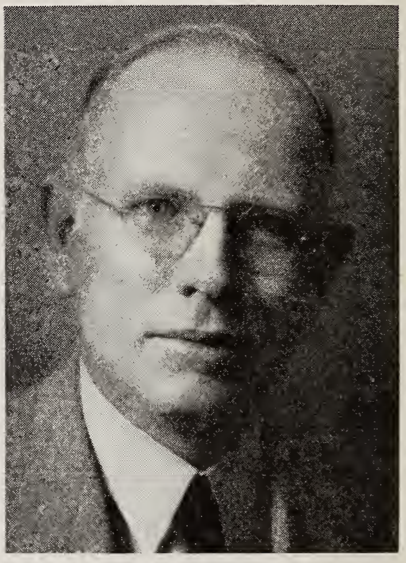
During the late World War the highways were inadequate. The present system is more serviceable but inasmuch as the peacetime and wartime requirements demand practically the same transportation arteries, there should be concentration on the development of the interregional and military network at once for the purpose of full military preparedness and the completion of the primary highway network. These roads built principally to meet commercial needs must meet military needs just as well.



DISPOSITION OF HIGHWAY FUNDS

The chart shows the current lack of uniformity in the distribution of motor vehicles funds and the large amount of diversion in many states.

The Bituminous Surfaced Highway of Tomorrow



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By title this paper is limited to the bituminous highway of tomorrow. It is further limited to a materials viewpoint by instructions from the Chairman. In this discussion the author has largely confined himself to the underlying soil and bituminous surface, and therefore, has not considered such items as width and alignment. It should also be pointed out that insofar as possible, the improvements in materials and manufacturing processes are based on scientific principles that are known today, but the author did not allow this to confine him. In some instances he soared off into the realms of pure imagination. Inasmuch as this paper is based on conditions in 1960, it is only fair to request that adverse criticism be withheld until that date.

The bituminous highway of 1960 will be designed and built on an entirely different basis from that of today. By that time the research specialists will have learned to apply nature's laws, and engineers will know how to put this knowledge into practical use. This will come about because the research specialist will know his own subject so well, that he can express himself in more simple terms, and because the field engineers will understand the fundamental sciences more thoroughly. As a result of this widespread application of scientific principles to road building, bituminous roads will be built at a fraction of their present cost. This large reduction in road building costs will make possible a much greater mileage of improved highways for the use of the myriads of motor vehicles. By that time the construction and maintenance of highways will cost much less than at present and the automotive research engineers will have greatly reduced the cost of car operation.

Thus the mad race will continue. Better cars and better highways at lower cost will result in more and more travel. (This will create demands for more highways, which when built will induce more traveling, resulting in demands for still more highways, and so on indefinitely.) Incidentally this will solve the unemployment problem, for combined with a low living cost and a delirium for touring, people will have neither the need nor the time for work. The luxuries as well as the necessities of life will be cheaply produced by machinery. By 1960 the people will be perfectly willing to let the machines do the work.

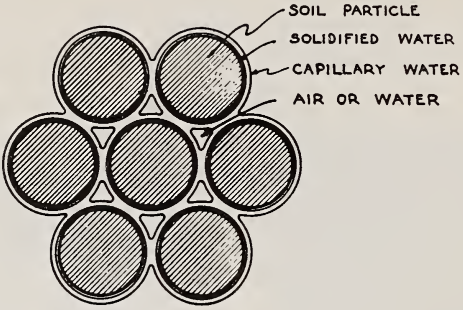
This great advance in low cost bituminous road construction will result mainly from two fundamental discoveries. One is concerned with water control and the other with an improved bitumen. This paper will be largely confined to a discussion of these two items.

WATER CONTROL

An early slogan in road building was "drainage and then more drainage." Based on the known fact that most soil foundations become less stable as the water content increases, it appeared to be basically sound to increase the expenditures for underground tile and deep side ditches to remove water.

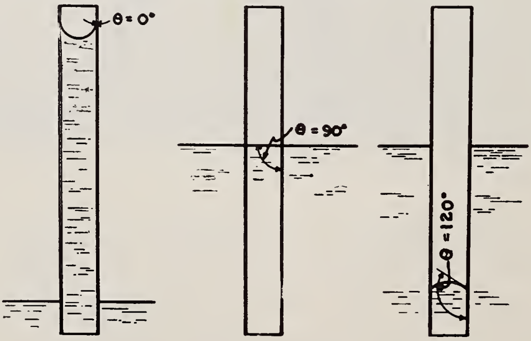
Slide 1.

About 1920 much money was wasted and many headaches were caused by the failure of underground drains to remove capillary water. The attempted cure for frost boils illustrated in this slide caused aches for the motorist in other places than the head. In this case tile was installed in gravel filled trenches to remove the gravitational water resulting from the thawing of ice lenses. This was accomplished, but the practical man overlooked the fact that ice lenses were nourished by capillary water. During the winter the road heaved everywhere but where the granular back filled trenches were located. Imagine the motorist's discomfort when bumping across such transverse trenches spaced 20 to 30 feet apart! It is well known that fine-grained soils may contain enough capillary water to cause instability several feet above the water table. It is usually impracticable if not impossible to lower the ground water level enough by underground drainage to remedy such a condition.



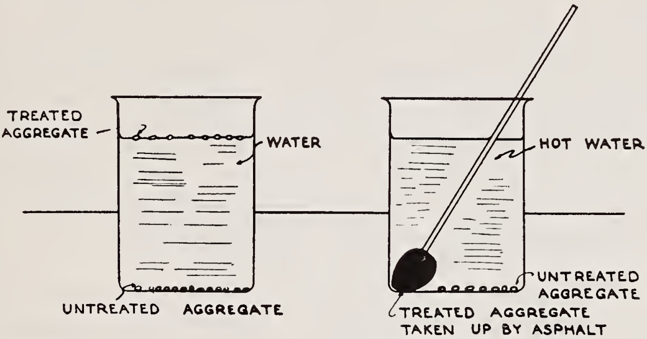
Slide 2

Water rises vertically against the force of gravity in fine grained soils because it is attracted by the soil particles. The attraction is so great that the very thin moisture film adjacent to the soil particle is under enormous pressure and has the properties of a solid. As the distance from the soil particle increases, the attraction decreases until finally it is no longer great enough to hold water. If the soil particles are large enough, taking into consideration the height above the water table, there will be a space between the films of cohesive water. These, as shown on the slide, may be filled with air or water. When this space is filled with water we no longer have these elastic films tending to pull the soil particles together. If the soil particles are very small as in clays, then it is possible that these spaces will be completely filled with capillary water. If the fine grained soil, such as clay, is dry, only the solidified water film remains, and the soil particles are held so firmly together, that the soil mass may have a strength greater than concrete. There is no need of explaining how unstable these soils may become when there is an excess of water. The problem in water control of fine grained soils, therefore, is to take advantage of the binding properties of thin water films, and to prevent any possibility of excess water.



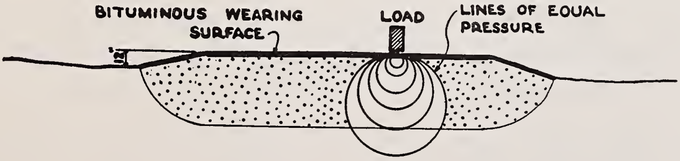
Slide 3.

The water drawn up in the glass tube at the left illustrates how water is pulled up into the soil pores by the molecular attraction which the particle has for water. The height to which water will rise depends on the degree of this attraction as measured by the contact angle, θ, and the weight of the water per unit of wetted surface, the latter depending upon the cross-sectional area of the tube or soil pores. As angle θ approaches zero, the height to which the water is lifted increases. If angle θ could be made 90°, there would be no capillary rise, and if it could be increased, say to 120°, in a soil, then the soil would remain dry even though it were under hydrostatic pressure, as it might be, in cuts through hills. In the center tube the angle θ is 90°, and at the right it is 120°. The basic requirement, therefore, is to change molecular compatibility into molecular incompatibility. This is done by introducing a substance which is attracted to the soil but which repels water. As some of the present day experts on stripping might say, the soil particle is changed from hydrophilic to hydrophobic.



Slide 4.

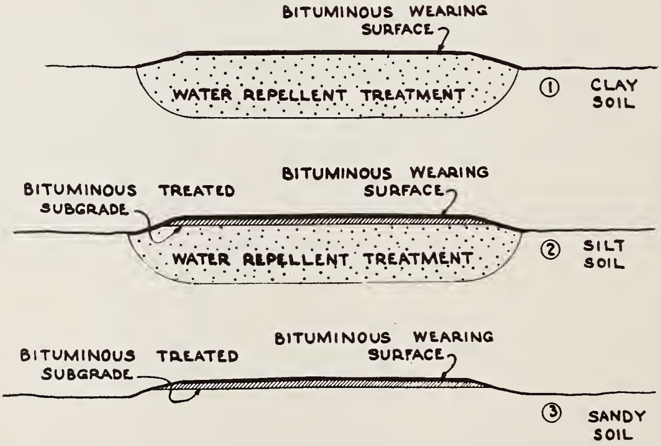
Perhaps this thought may be made clearer by this slide, illustrating a demonstration which the writer recently saw made in the chemical laboratory of a company manufacturing aggregates for roofing. This company wished to stain quartz aggregate particles different colors, such as red and green. The quartz, however, had a greater molecular attraction for water than it had for the asphalt, and so when water was present it displaced the asphalt. It was found, however, that this relative molecular attraction could be reversed by spraying the hot aggregate with a very weak solution of a certain substance, which, upon drying, became insoluble in water. Such treated aggregates when coated with asphalt would not strip in the presence of water. In the beaker at the left, treated particles up to 1/10 inch size repel water to such an extent that they float on the water. In the beaker at the right, treated (green) and untreated (white) aggregate particles were placed in water and heated to boiling. A ball of soft asphalt attached to a glass rod was then stirred into the aggregate. In a very short time the green (treated) particles of aggregate were incorporated into the asphalt, while the white (untreated) particles remained free.



Slide 5.

The ideal construction, as shown on this slide, would consist of a roadway having shallow ditches for safety, and with water repelled from the pores of the underlying soil by some such treatment as previously described. Some further discoveries must be made before this principle can be applied to a fine grained soil in order to enable us to build stable roads with high ground water levels. We may reasonably expect, however, that eventually research men such as Winterkorn and McLeod will find a method to treat cheaply fine grained cohesive soils in place. Possibly this will be done by spraying air dried soil with a cheap solution or suspension of some petroleum derivative, which upon drying will repel additional water yet utilize the cohesive forces of the existing thin films. It may well be that the very subsoils which are now the most troublesome in low cost road construction, such as the A-6 and A-7 types, will be the preferred soils of the future. The required depth and intensity of the treatment for any soil would depend upon the magnitude of the load and the distance from the application of the load. As indicated in this slide, the stress in the soil decreases as the distance from the point of application increases. It is reasonable to expect that Housel and others who are studying soil mechanics and its relation to the successful construction of bituminous surfaces will have this all worked out. There will be no guessing or perhaps for engineering one should say, no estimating. The principal functions of the bituminous surface will be the same as now, i. e. to shed water and to resist traffic abrasion.

This type of treatment would prove ideal for silt soils where capillary water proves so troublesome, especially in northern climates where frost heaving occurs. The application of the water repellent material would be simpler than in the case of the cohesive soils. The addition of some bituminous material to supply cohesion would be required, however, particularly in the upper portion of the treated subgrade.


BITUMINOUS WEARING SURFACE
WATER REPELLENT TREATMENT
① CLAY SOIL
BITUMINOUS TREATED SUBGRADE
BITUMINOUS WEARING SURFACE
WATER REPELLENT TREATMENT
② SILT SOIL
BITUMINOUS TREATED SUBGRADE
BITUMINOUS WEARING SURFACE
③ SANDY SOIL

Slide 6.

This slide illustrates the different types of bituminous road construction for the three typical soils—clay, silt and sand. Modifications will be made for mixtures of these soils. You will observe that on:

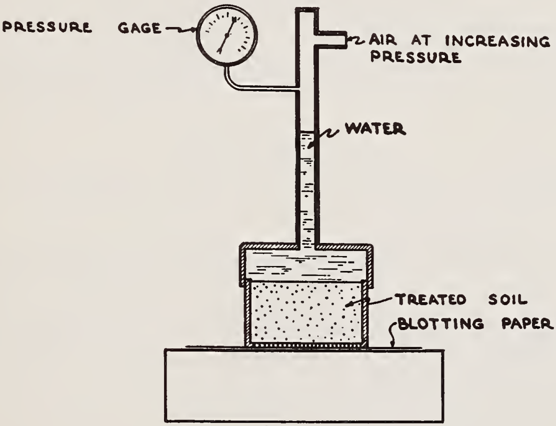
1. Clay soils—The thin bituminous surface is laid directly on the treated soil. The densified, dry, water repellent, clay soil of high cohesive strength, would have ample load carrying capacity.

2. Silt soils—The underlying silt soil in addition to receiving the water repellent treatment has been mixed with bituminous material near the surface. The silt particles are too large to be cemented together by the solidified water films, and so the necessary cohesion must be supplied

by adding bitumen. As with the clay soils, the bituminous wearing surface is thin.

3. Sand soils—This type of soil does not become unstable from capillary water. If there is an excess of gravitational water present, it can usually be easily drained. For this reason the soil is not given the water repellent treatment, except near the surface, where bitumen is added to increase cohesion as just described for silts. The bituminous treated sand foundation must be of sufficient width and thickness so that there will be no lateral displacement of the underlying sand. The actual wearing surface is again very thin.

For most types of soils, no other soils or aggregates will be hauled to the road for base construction. It is anticipated that there will be some selection and densification of soils during grading but that the grading will be carried out primarily to obtain suitable vertical and horizontal alignment.



Slide 7.

This slide illustrates a test for determining the effectiveness of the treatment. It shows an apparatus somewhat similar to the drainage indicator devised by Hogentogler. Water will be applied to a consolidated sample of treated soil at a pressure which increases uniformly with time. The time required for the first drop of water to penetrate through the specimen will be a measure of the degree of waterproofing obtained. Some such test can be used for determining the amount of chemical necessary to treat any soil to meet the road conditions.

ASPHALTIC MATERIALS

At the present time after reviewing the progress that has been made during the past 20 years, it scarcely seems possible that by 1960 the asphaltic products and the tests for determining their quality will be entirely changed. If we place ourselves in 1960, however, we find that the executives of the petroleum industry, as a result of pressure from competing materials, have decided to establish a central research laboratory and to cooperate with the consumers as outlined by Director Crum of the Highway Research Board at the 1938 Montana National Bituminous Conference. The results were far beyond what Crum apparently anticipated. The research men started to develop better tests for measuring quality, but became sidetracked and discovered a way to make much better bituminous material. Then they developed new tests to measure the quality of this improved product. The executives of the industry were quite elated when they not only recovered their waning business but increased it to such an extent that the heavy hydrocarbons were no longer a surplus commodity. As a result, they were able to increase the price and make greater profits. The discoveries which led to the manufacture of this greatly improved asphalt came just in time to supplement the discoveries in water control in base construction and make possible this greatly improved, low cost, all-weather highway.

It is not possible in 1939 to describe the processes by which this new asphalt is manufactured. The heavy hydrocarbons in asphalts are known to be chemically very complex. The improved asphalt of 1960 has been developed after much painstaking research and eventually a little luck. It happens occasionally that the trained research specialist, during his work, observes some phenomena which he had no reason to anticipate and then, as a result of concentrating upon the reasons for this unexpected phenomena, some of the most important discoveries are made. We often hear of how great discoveries are made accidentally, but one must not overlook the fact that a trained scientific mind observed and studied the accident.

This new asphalt is superior to the 1939 asphalts in many respects, such as:

1. Only one grade or viscosity of asphalt is ordinarily used. This is a thin bitumen which does not change in consistency with temperature. It is not necessary to heat it or thin it in any other way to coat the aggregates. By adding a certain admixture during the mixing, the final viscosity can be made whatever is desired. The interval of time before the asphalt reaches the final consistency can be controlled so as to give time for placing and consolidating. Weather does not delay the work to any extent when such a bitumen and treated water repellent aggregates are used.

2. After placing, there is no bleeding or pushing due to softening of the asphalt during hot weather, nor need there be any cracking or disintegration during cold weather.

3. The asphalt, when placed, is very adhesive and has elastic properties similar to rubber. There is practically no deterioration from exposure to sunlight, air and water.

In addition to having all of these desirable properties, this asphalt is so easily and cheaply manufactured that there is no tendency for the producers to ship inferior materials.

CONTROL TESTS

Tests have been developed which are a measure of the physical properties that the asphalt, as well as the resultant product after the admixture was added, should have. There are no longer any tests to show ancestry or manufacturing processes. Aside from some tests to assure safety in handling, the tests are made to measure viscosity, time of set, adhesiveness to treated aggregates, durability, and elasticity. Time will not permit a discussion of tests for measuring these properties.

SECTION I

Research as Applied to Construction and Maintenance of Bituminous Surfaces

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Current Research Problems in the Construction and Maintenance of Bituminous Surfaces

By

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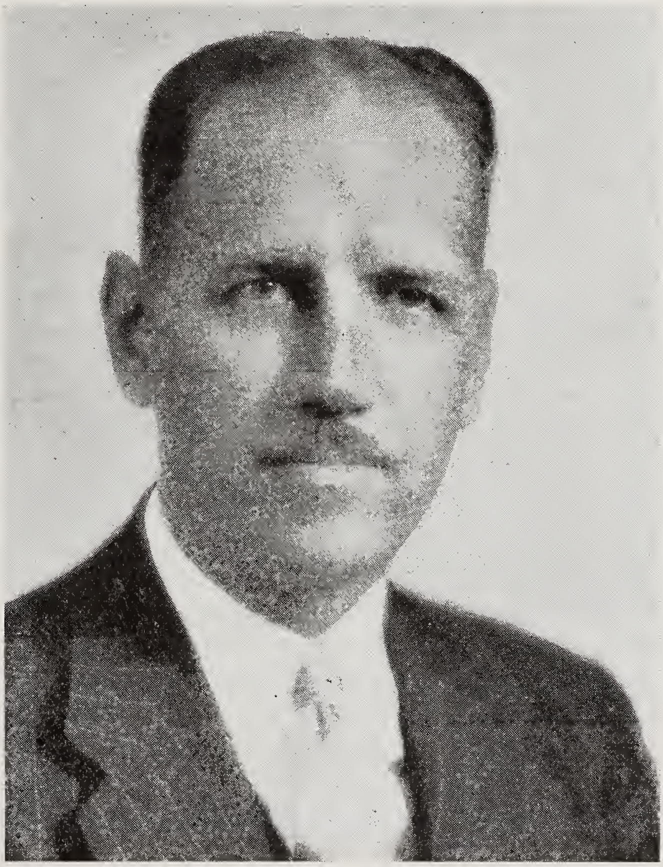

A review of the proceedings of this conference for last year shows that research in bituminous road construction emphasized a number of problems which are closely allied with soil and soil mechanics. Subgrades, drainage, soil stabilization, physical properties of granular mixtures, and mechanics of flexible surfaces were subjects of primary interest. When the members of the research section were asked this year to select the research problems of greatest immediate importance in this field it is interesting but not surprising to find that interest has shifted back to some extent and at this time appears to be focused more on bituminous materials themselves and definitely related properties of bituminous mixtures as they affect service behavior.

One might assume that the other problems mentioned require a period of trial and assimilation and that members of the conference, having been made fully aware of their importance, are now emphasizing other phases of bituminous road construction.

At any rate the papers submitted to the research section this year may be assembled into the two main divisions of bituminous materials and bituminous mixtures, excepting one paper on soil stabilization, which is devoted merely to a discussion of recent developments. The papers on bituminous materials, seven in all, are concerned with an interpretation of current tests of bituminous materials and methods of procuring more permanent and satisfactory service behavior by this means. Four of the papers are devoted to the change in characteristics of bituminous material both during construction and in the pavement and with methods of improving this aspect of bituminous construction. The other three papers deal with slightly different aspects of bituminous materials but have the same general objective. Four papers have been classified under the head of bituminous mixtures and treat the subject from the standpoint of current tests and their relations to service behavior. One paper in this group is devoted to a general summary of the present status of design and construction.

* * *

In this paper on the "Hardening of Asphaltic Materials", Mr. Juhlin cites various authorities who have been studying the drop in penetration of asphalts during the process of mixing and placing. In a general discussion of this whole subject he points out that permanence is the objective to be approached but can probably never be attained in any type of pavement. Hardening of asphalts is the primary cause of cracking and subsequent deterioration, aside of course from faulty subgrades, drainage, and other factors which are not considered as a part of this discussion.

In attempting to set limits to which the penetration may drop before becoming dangerous, Hubbard and Gollomb have selected approximately 30. On the other hand, Powers has reported penetrations as low as 15 without failure. The biggest drop in penetration occurs during mixing and placing. Bateman and Lehmann have found drops of a 46 penetration asphalt to 40, 36, 35, 33 and 24 at the respective mixing temperatures of 250, 300, 350, 450 and 500° F. Rader records a case of a 56 penetration asphalt which had dropped to 34 in a sheet asphalt pavement eight years old. He cites a second case of a pavement eleven years old with a penetration drop from 42 to 13. Raschig and Doyle have recorded a number of instances of lower penetration of the asphalt extracted from various pavements. Covert reports a drop in penetration from 45 to 24 in a pavement five years old and notes that the largest loss occurred in the mixer box. He found for an 80 penetration asphalt and a mixing time of 45 and 15 seconds a penetration loss of 12 to 15 per cent. Miller, Hayden and Vokac examined a large number of pavements, classifying them into three groups, A, B, and C, the first in which cracks developed, the second which had neither cracked nor shoved, the third in which the mixtures shoved. They found that the average hardening expressed as penetration loss on the recovered bitumen was 63, 43 and 29 per cent respectively for average ages of 10, 9, and 14 years.

In a discussion of the importance of mixing temperatures Mr. Juhlin notes that the usual specifications allow heating of the aggregate to 375° F., the asphalt cement to from 250 to 350°, and call for placing the mixture at not less than 275° F. He notes that the hardening effect of these temperatures is augmented by the high void content of the uncompacted mixtures which facilitates oxidation. Hardening to a large extent is due to oxidation and subsequent increase in asphaltenes. It is noted that the high temperatures in thin films are the important factors in speeding up oxidation. Experimental data which verifies this contention is furnished by the results of air-blowing of soft residual asphalts. Using a soft residual asphalt with 100° F. softening point by the ball and ring method the following results were obtained.

Softening point, °F., (B & R)	101	113	118	140	178
Penetration at 77° F. (100 g., 5 sec.)	179	112	85	46	24
% insoluble in 86° naphtha (% asphaltenes)	12.3	18	19.1	23.5	29.2

Hubbard and Gollomb using asphalts from nine sources prepared sheet asphalt mixtures in which oil asphalts of 50 penetration showed a penetration drop of from 19 to 36 per cent after being exposed in loose layers at 325° F. for one-half hour. Skidmore and Abson report that irrespective of source 10 to 20 points in penetration are lost under good plant practice as indicated by laboratory tests.

Another type of hardening which is also a factor is generally spoken of as ''age hardening'' or ''thixotropy'' and is an effect that is destroyed by heating or kneading the asphalt. Traxler and Schweyer investigated this change and developed an aging index as well as ascertaining the temporary character of the change.

Remedies which are being generally introduced are a careful control of mixing temperatures, mixing time, and the elimination of abnormally high placing temperatures. The general trend toward the use of softer asphalts to prolong the life of the pavements is mentioned. Mr. Juhlin states that little information is yet available on hardening of asphalt in the pavements but notes that several states, including Michigan and Ohio, are collecting data of this character. Studies of this nature have been greatly facilitated by the Abson recovery method which makes possible more reliable results on the extracted asphalt.

* * *

In a paper on the ''Changes in the Characteristics of Paving Asphalt Cements and their Relation to Service Behavior'' Mr. Schaub and Mr. Parr of the Michigan State Highway Department present the results of several years' study of a number of pavements included in a comprehensive survey. These results are of particular interest inasmuch as Michigan, in 1935, was the first state to set up a specification for the ductility and penetration of asphalts recovered from the pavement after laying, with a penalty clause if the changes exceeded a certain specified limit and provision for complete rejection if they exceeded certain minimum values.

Subsequently the Department started a number of pavement surveys to be carried over a period of years in order to obtain basic data for future reference. Seven bituminous concrete pavements laid in 1934 and 1935 were first selected. Both straight run petroleum asphalts and blends of equal proportions of petroleum asphalt with native asphalt were included in the study. From these projects it was found that the average drop in penetration during mixing and placing was approximately 20 per cent with subsequent drops of 20 per cent, 10 per cent and 5 per cent for each of the succeeding three years. Changes in ductility were much more erratic than in the case of penetration but after three years at least four projects have shown a serious decrease in ductility.

In 1936 an experimental bituminous concrete project was selected to provide data under more carefully controlled conditions. This project included five straight run petroleum asphalts and one native asphalt and four blends of equal proportions of petroleum and native asphalts, making nine sections of pavements for detailed study. The average drop in penetration of these materials during mixing and placing is 25 per cent, ranging from 2 per cent to 47 per cent. Subsequent penetration losses amounted to a total of 50 per cent after two years, the first year being 15 and the second year 10 per cent.

Two special cases are of particular interest, a blend of one high penetration native asphalt with a fluid flux showed at 47 per cent drop during construction which is compensated for to considerable extent by much slower change during the succeeding two years. The present data indicate that at the end of approximately five years this particular material should have about the same penetration as the others. It seems reasonable that the high evaporation normally to be expected in this particular case may account for the abnormally high decrease in penetration during mixing and placing. One straight run petroleum asphalt showed only a 2 per cent drop during construction and the aging effect is taking place at about the same rate as the other asphalts being studied.

It is noted that there is no significant difference as far as present results are concerned between the straight run asphalts and the various blends that were used. In this particular project the period of time has not yet been sufficient to give evidence of any differences in service behavior.

A third series of projects being studied consists of seven sheet asphalt pavements laid during the period from 1934 to 1937. On these projects the drop in penetration during construction was 20 per cent, about the same as for the bituminous concrete pavements. The changes, however, taking place after the pavements were laid were found to be considerably different. In the first place, the decrease in penetration and ductility was considerably more erratic and on the average, these decreases were substantially less in the sheet asphalt pavements. Thus in the first year the drop in penetration was 11 per cent and only 3 per cent for each of the two succeeding years of the three-year period available at the present time. Possible explanations are offered in that the sheet asphalt mixture is more impermeable preventing the access of air to the mixture and decreasing the subsequent oxidation. Another possibility offered was that the sheet asphalt pavements being in more populous districts and carrying heavier traffic are affected by accumulation of oil waste which acts as a fluxing agent and offsets the usual hardening. The results of this particular portion of the study are still too limited to draw any final conclusions but they do provide some rather definite indications.

The only pavements which have been in place long enough to give any indication of the correlation between changes in the physical characteristics of the asphalt and service behavior are found in the first group of bituminous concrete pavements laid in 1934 and 1935. Two particular projects have shown definite signs of distress. Cracking appears in two years and at the end of five years the pavements were raveling rather badly. The total penetration drop to date on these projects is 63 and 64 per cent and the average ductility dropped to 5 and 12 cm. from an original of 100+. While these results appear to be very significant, it must also be noted that other factors enter into the picture.

Two other projects of the same group have approximately the same drops in penetration and ductility but neither of them have given the same evidence of failure. There appear to be compensating factors in the mixture design which are responsible for this difference in behavior. The first two projects which have shown failures differed materially in the percentage of coarse material. There was 63 per cent of material retained on the No. 10 sieve in the case of the first two projects as compared to approximately 55 per cent on the two which had not failed. There was also approximately one-half per cent higher bitumen content in the two latter projects. Experience has indicated that more than 50 or 55 per cent coarse material, retained on No. 10 sieve, results in a mixture which is more permeable to air and perhaps deficient in mechanical strength to resist shrinkage stresses, and it also appears that this type of mixture should carry a minimum of bitumen on the order of 6 per cent.

Several remedies are suggested to compensate for the changes in physical characteristics which have been noted, some of which have already been put into effect in the Michigan specifications. The use of seal coats to protect the open mixtures is already rather general. In 1936 the maximum size of aggregate in bituminous concrete mixtures was reduced from ¾ to ½ inch. At the same time the desirability of the use of softer asphalts has been recognized by changing the penetration range of from 50 to 60 to a range of from 65 to 75.

Some other supplementary data is included in this paper having to do with control tests of mixing temperature and time. Parallel tests were run in a laboratory pug mill and the conventional paving plant with excellent correlation. It was found that the drop in penetration was directly proportional to the increase in temperature in a range of 325° to 480° F. Mixing time was also varied keeping the temperature constant at 340° F. plus or minus 10. It was found that the big drop in penetration comes in the first 30 seconds and amounted to approximately 30 points whereas the drop was only 10 points in the next 150 seconds. Thus it appears that mixing temperature is considerably more important than the mixing time.

Detailed data are included with the paper in the form of tables and charts.

* * *

The paper by Professor H. E. Phelps of the State College of Washington on "An Evaluation of Recent Trends Toward the Use of Softer Asphalts" summarizes the current opinion on this subject. Investigators have concluded that the old specification which provides a penetration range of from 50 to 60 may profitably be increased at least 10 points to a range of from 60 to 70 and possibly as high as from 100 to 120.

Professor Phelps quotes Buchanan who states that current studies indicate the desirability of using softer asphalts in both hot and cold climates. The fact that in hotter climates the asphalts suffer greater changes in penetration and ductility eliminates the necessity for the use of harder asphalts. It is noted that the major resistance in a bituminous mixture is due to aggregate stability and does not depend primarily upon the consistency of the asphalt. The fact that satisfactory results are being generally obtained in hot climates with the use of liquid asphalts is supporting evidence for this suggested change.

The use of softer asphalts allows a decrease in mixing temperatures and thus eliminates the major source of change in characteristics. That these trends are not unsound was further indicated by the successful use of liquid asphalts of 150 to 200 penetration corresponding to an SC-6 grade in hot-mix pavements. Oregon also reports an RC-4 liquid asphalt in place of the paving asphalt usually used in bituminous concrete. There appears to be an almost unanimous opinion among paving engineers that the trend discussed by Professor Phelps will effect a definite improvement in bituminous construction practice.

Summarizing these opinions which are borne out in all three of the first papers that have been presented, the following conclusions by Hubbard and Gollomb are quoted by Professor Phelps:

1. Extreme hardness of asphalt in an asphalt pavement is conductive to cracking.
2. Contact with air is a principal cause of hardening of asphalt.
3. All asphaltic products harden progressively and with appreciable rapidity when continuously exposed in thin films to the action of air.
4. Rapidity of hardening is a function of film thickness, temperature and time — the thinner the film, the higher the temperature and the longer the period of contact with free air, the greater will be the degree of hardening.
5. Thin films of all liquid asphaltic road materials, as well as asphalt cements may be expected to harden to low penetration

brittle asphalt within a few years' time if not adequately protected from contact with free air.

6. At hot-mix paving plants, asphalts may be expected to harden appreciably during their combination with mineral aggregate under customary plant operation.

7. From data at present available it appears that the following expectancy of cracking due to hardness of the asphalt cement present in the pavement, exists in climates where freezing temperatures are common through the winter:

   (a) Asphalt less than 20 penetration—bad cracking.

   (b) Asphalt higher than 30 penetration—no cracking.

   (c) Asphalt 20 to 30 penetration—uncertain.

8. An asphalt of 50 penetration, irrespective of source, may very likely be reduced to as low as 35 penetration during preparation of a hot-mix thus bringing it dangerously near the limit of penetration at which the pavement may be expected to develop some cracks.

9. In hot-mix construction the softer the original asphalt the less it is likely to harden dangerously during the mixing operation. For such construction an asphalt of 60 penetration or softer is strongly indicated as a means of preventing hardening to a dangerous extent during the mixing process.

10. Progressive hardening of asphalt films in a dense thoroughly compacted paving mixture is exceedingly slow. If, however, the mixture is readily permeable to air, dangerous hardening may occur within a relatively short period.

11. To insure long life, prevent cracking and possible disintegration in an asphalt pavement or wearing course, due to hardness of the asphalt, the following five rules should be strictly observed:

   (1) Use as soft an asphalt as possible without reducing stability below the minimum required to prevent displacement under traffic.

   (2) Use as high a percentage of asphalt as possible, without reducing stability below the minimum required to prevent displacement under traffic. In this way film thickness is increased to the maximum practical extent and air permeability decreased.

   (3) At paving plants prepare mixtures at the lowest practicable temperatures.

   (4) Compress all asphalt paving mixtures thoroughly so that they will be as impermeable to air as possible.

   (5) Seal coat all compressed mixtures which are readily permeable to air, and thus prevent free circulation of air within the mixture.

Doctor Winterkorn of the University of Missouri discusses ''Quantitative Methods of Measuring Adhesion of Bituminous Materials,'' a subject which he is well qualified to present. In the introduction of the subject he notes that quantitative methods of measuring adhesion consist primarily of the evaluating stripping phenomena. Two major types of stripping are considered, first, when the surface of the mineral aggregate is washed clean due to displacement of the bituminous film by water, and second, when the surface is left with a thin coating of deteriorated bitumen suggesting the oxidation of the thin bituminous film.

The usual conception of adhesion pictures the aggregate as having a clean mineral surface and overlooks the fact that other substances, such as water and air, are usually intimately bonded to the surface of the aggregate. These films are not removed at ordinary ranges of temperatures and humidity and the water may thus be trapped inside the bituminous film and under certain conditions become free water and cause stripping. Thus Doctor Winterkorn quotes the necessity of observing these phenomena by the use of soil colloids with the highest specific surface inasmuch as the quantities of moisture are otherwise too small to be measured directly.

Baver and Horner investigated five such colloids which were allowed to absorb water in humidified atmosphere and then dried at 60° C. The total absorbed water varied from 7 to 25 per cent of the dry weight of the soil colloid of which 75 per cent could be set free at the drying temperature. The significance of this result lies in the fact that as such water became trapped by a bituminous film and then freed by a subsequent increase in temperature a serious tendency toward stripping could be anticipated.

Further study of four other soil colloids by Baver and Winterkorn under controlled vapor pressure and relative humidity gave an adsorption of 21 to 23 per cent of the dry weight at 30° C. with only 1 to 3 per cent of the adsorbed moisture retained at 80° C. Thus large quantities of water may be adsorbed at a low mixing temperature and under conditions of high humidity, and this moisture set free at a higher pavement temperature. These studies indicate the desirability of utilizing mixing temperatures which would eliminate the adsorbed moisture before the mixture is laid in the pavement. Drying aggregate and allowing it to cool in a humid atmosphere would obviously not be of much benefit unless moisture control is also introduced in the cooling chamber. In order to measure adhesion effects for controlling the behavior of the finished pavement the aggregate must be allowed to come to equilibrium with respect to adsorbed moisture at a definite temperature and vapor pressure which should comply with actual field conditions. If these conditions are controlled, any of the widely used wash tests provide a valid measure of the ability of the bitumen to adhere to the mineral aggregate. These data also throw some light on the moisture limits which should be obtained in a mixture before it is placed in the road surface and compacted. The percentage of moisture should not exceed the moisture adsorbed by the aggregate at the highest probable temperature in service.

These observations indicate why stripping trouble has originated with cold mix construction but was seldom heard of when hot-mix was the prevailing method of construction. After the coating has been properly done, as indicated in the previous discussion, the danger of stripping is minimized because surface energies of water and bituminous materials are not enough different to allow the moisture to diffuse through the bituminous films except when these films are very thin or discontinuous. This is probably one of the greatest sources of difficulty now being encountered in soil stabilization where bituminous films are extremely thin and mixing methods require considerable quantities of moisture for dispersion, which moisture is not usually brought down to the required limits before compaction. Thus in cold mix pavements the physico-chemical character of the aggregate becames extremely important. Those aggregates are preferred which do not show appreciable differences in the moisture adsorbed at high and low temperatures.

There are certain other phases of stripping phenomena which must be considered in a somewhat different category. In this connection Doctor Winterkorn refers to coating of wet aggregate with bitumens or the use of emulsions. The degree of coating is sometimes used as an index to the character of the adsorbed moisture film and considered to be a measure of tendency for stripping. The author indicates some objections to this procedure with one noticeable exception. The exception occurs when the character of the surface of the aggregate is changed by the addition of chemical reagents which provide favorable linkage between the mineral surface and the bitumen. This is the method developed by McLeod and has been presented at previous meetings of this conference. In this case it happens that the quantity of moisture present is not a controlling consideration and the coating test is regarded by Doctor Winterkorn as applicable. On the other hand, the coating test in the presence of excessive moisture cannot be used to replace the wash test on previously coated aggregate as a measure of adhesion. To obtain a reliable measure of stripping in wet coated aggregate the material to be tested should come to the same moisture content to be used when the mix is compacted.

In another respect the wash test is unsatisfactory as it does not accurately represent the behavior to be anticipated in the actual pavement. Tests which appear to be more applicable in this connection are the widely used swell tests, stability test before and after saturation with moisture. There is very little agreement as to the method of saturation which should be used, whether is should be capillary absorption, immersion, storing in highly humidified atmosphere, or immersion in partial vacuum. Doctor Winterkorn suggests that this is a subject which should be undertaken as a comprehensive study by some national agency.

While the author devotes most of his time to the importance of adsorbed moisture, he makes note of the comparatively new line of investigation which has to do with the effect of adsorbed films of air or other gases, a subject which is being studied by Powers.

Summarizing it can be stated that the best available methods for testing the resistance to stripping of bitumen coated mineral are different modifications of the wash test in which the coated mineral particles are shaken in the presence of water. These methods furnish quantitative data if the conditions to which the aggregate is subjected before, during and after the coating are standardized as well as the actual test procedure. This standardization must be of a kind to insure definite equilibrium between the surface of the mineral and the gaseous or liquid phases with which it comes in contact before being submitted to the action of water during actual testing. To be of practical value the standardized procedure should represent as closely as possible the condition obtained on the road.

* * *

Mr. Zapata's paper on the "Interpretation of Current Tests on Bituminous Materials in the Light of Recent Research" gives a summary of the results of the comprehensive investigation made jointly by the author and Professor O. A. Hougen of the Department of Civil Engineering of the University of Wisconsin. The complete results of this investigation were published in a bulletin of the University of Wisconsin, College of Engineering. The purpose of this report is to present a summary of the correlation between composition expressed in specific terms and the results obtained by current tests on asphalts of the slow-curing type. Although his report is limited to this type of material, some of the findings may also apply to other types of asphalt.

Recognizing that this subject is one which has been given a great deal of attention, Mr. Zapata attempts only to present data which does not appear elsewhere for the purpose of inviting attention to the possibility of new interpretations. The original scope of the investigation may be summarized as follows:

1. To find an explanation for the differences in service behavior of slow-curing asphalts meeting the same specifcations.
2. To examine critical specifications for slow-curing asphalts in order to determine the value of the various requirements and their relation to service behavior.
3. To determine the influence of technical and physical properties of slow-curing asphalts upon their adhesion and resistance to stripping.
4. To study the effect of composition on the properties of slow-curing asphalts.

Because of the difficulty of obtaining samples of systematically varying composition, the investigators in this instance obtained large samples of five slow-curing asphalts from differene sources and produced by different methods of refining. By means of solvents the samples were then separated into fractions identified as asphaltenes, resins, and oily constituents, and systematically recombined into synthetic samples which were tested for various properties. The five asphalts used in making up these synthetic samples may be listed as follows:

1. An uncracked, steam-refined Texas residue, naphthenic.
2. An uncracked, steam-refined Mid-Continent residue, paraffinic.

3. A slightly cracked residue, aromatic.
4. An uncracked Mexican residue, naphthenic.
5. A highlycracked residue, aromatic.

The fractions produced by separation represent three distinct ranges of molecular weight: About 2,000 for the asphaltenes, about 1,000 for resins, and about 500 for the oily constituents. To determine whether or not possible changes had occurred in the fractions, original samples and synthetic samples of similar composton were compared by identical tests. All properties with the exception of viscosity at low temperatures were found to be alike and this discrepancy was attributed to age, the original samples being tested several months after the manufacture while the sysnthetic samples were tested shortly after they were made.

The characteristics of the materials which were tested were the specific gravity, the viscosity, percentage of residue of 100 penetration, ductility of residue of 100 penetration, solubility in naphtha, fluidity factor, spot test, resistance to stripping, and the laboratory weathering test. The results of the tests will be summarized, being considered in the previously indicated order.

It was found that the specifc gravity of the blended samples was a simple arithmetic average of the product of the percentages and specific gravity of each constituent. The cracked residues showed the highest specific gravities in agreement with the low hydrogen content while the products from the Mid-Content crude showed the lowest specific gravities, also in agreement with the paraffinic nature and high hydrogen content of the crude. The Mexican and Texas crudes showed a striking similarity.

In attempting to establish correlations with other properties the most striking correlation was found between specific gravity and resistance to strippng. Increase in resistance to stripping was found to be closely related with the increase in specific gravity up to a value of 1 regardless of the bitumen or aggregate used. The results obtained in making approximately 5000 tests indicate that specific gravity is an important factor in evaluating resistance to stripping.

Viscosity was measured at three temperatures, 104°, 122°, and 194° F. by the rise in a capillary tube method. The results were studied not only as expressions of viscosity but in terms of viscosity index of Traxler, the viscosity-temperature-susceptiblity index of Nevitt and Krchma, and the viscosity-temperature-slope index. It was found that the viscosity is influenced greatly by the asphaltene content and to a lesser extent by the resin content. Generally speaking, the viscosity was about doubled by each 5 per cent increase in asphaltenes and a 10 per cent increase in resins. The cracked residues showed low viscosity following closely changes in composition. With similar composition the product of the Texas and Mexican crudes showed much higher viscsoities while the product of the Mid-Continent crude showed the least agreement between composition and viscosity. None of the three indices mentioned were found to be adequate in identifying the source of the crude from which the samples were prepared. Viscosity susceptibilities are greatly influenced by the asphaltene content. In correlating viscosity values with specific gravity and resistance to stripping, it appears that the viscosities now in current use are low.

The percentage of residue of 100 penetration was found by the method of the A.S.T.M. Serial Designation: D 243-36 as distillation tests have not worked out satsifactorily for the types of slow-curing asphalts which were used in Wisconsin. The method of determining the percentage of residue of specified penetration was correlated with field studies made by periodical examination of the bitumen extracted from a specially chosen project in the State Trunk System.

After these examinations were made for sixty weeks of service the following conclusions have been reached.

1. Evaporation of the oily constituents present in the original asphalt accounted chiefly for the changes in consistency occurring in the bitumen in the road.
2. At the end of sixty weeks the consistency of the bitumen in the road approached that corresponding to a penetration of 150. After evaporation took place the gain in consistency was found to be gradual and it was calculated that an appreciable period of time would elapse before a consistency of 100 penetration would be reached.
3. The changes that take place in evaporating a sample of slow-curing asphalt to a predetermined consistency are similar to the changes that take place in the road surface. Both are produced chiefly by evaporation. Thus the test is of value because it makes possible the performance of tests which may serve as a measure of the properties of the bitumen found in the road after a period of service.

The value of the ductility of residues of 100 penetration does not appear to be great. The laboratory of the Wisconsin State Highway Commission has tested literally thousands of samples for ductility at 77° F. and found no correlation between results of tests and service behavior. Furthermore, this property does not appear to correlate with other properties and at present its significance is not clear.

A comparison of the results of solubility in naphtha and hexane showed the solubility in naphtha was slightly greater, and it is believed that the naphtha precipitates a small fraction of resins along with the asphaltenes.

Originally the fluidity factor was intended to identify materials of naphthenic nature as it is believed that these products in general have higher durability. Otherwise the test was not intended to be a means of identifying good from bad asphalts. In these studies it was found that regardless of the composition of the blends in the series of cracked asphalts, they have fluidity factor values of lower than 130 and a great many have negative values. The product from the Mid-Continent crude showed a large range of compositions in which the fluidity factor values range from 130 to 150. The product from the Mexican and Texas crudes showed fluidity factor values greater than 150 over the entire range of compositions.

While the Oliensis spot test has undoubtedly filled a need in asphalt technology, the results of these tests indicate that it is possible to obtain positive spot tests on materials that have not been overheated, cracked, blown, or which do not have wax, sludge, or incompatible fractions. In the combinations from the products of the Texas and Mexican crudes it was found that the former had compositions of 40-50% asphaltenes, 20% resins, and 60-100% oily constituents which showed positive spot tests. Likewise the combinations from the products of the Mexican crude had compositons of 0-45% asphaltenes, 0-20% resins, and 45-100% oily constituents which showed positive spot tests.

These compositions were homogeneous in the sense that neither in their original conditions nor at any time during the performance of the test did any of the samples show separation or precipitation of fractions. The Mid-Continent showed a definite area in which all compositions gave positive tests and within which a number of compositions showed nonhomogeneity, while the cracked residues as expected showed positive spots throughout the entire range of compositon. Possibly the areas of positive spot tests for the Texas and Mexican crude products do not include compositions of commercially available products and therefore the spot test would still be significant in its practical application.

The resistance to stripping was measured by a means of modified Nicholson procedure and by the Reidel and Weber method. Three types of aggregate were considered: Quartzite of acid nature, dolomitic limestone of basic nature, and glacial gravel of a mixture of sedimentary, metamorphic and igneous aggregates. It was found that asphalts of paraffinic nature adhere better to the acid aggregate while asphalts of naphthenic nature adhere better to the basic aggregate. The cracked residues showed particularly good adhesion to the acid aggregate. The difference in resistance to stripping toward aggregates of this nature disappeared when the specific gravities of the bitumen exceeded values of 1. It was found that resistance to stripping from aggregates of one size or of a limted range in size does not correlate with the resistance to stripping from aggregates containing a range of sizes found in actual mixtures.

The results of the laboratory weathering tests were disappointing because no correlation could be found between laboratory tests and behavior in service. Neither the appearance nor the asphaltene content followed changes taking place in the road. The laboratory weathering cycles were planned to simulate weathering conditions found in Wisconsin. Thin films of bitumen of uniform thickness were applied to aluminum panels and exposed four hours to ultra-violet light, nine hours to a spray of lake water, two hours to freezing temperatures, and then allowed to dry for nine hours at room temperature. It was concluded that laboratory weathering tests involving exposures to ultra-violet radiation, to freezing, and water spray do not duplicate results obtained in service.

* * *

In a paper on "The Efficacy of the Oliensis Spot Test and Slightly Cracked Asphalts" Mr. Oliensis presents the results of a careful and comprehensive investigation of seven asphalts distilled from an Oregon basin, Wyoming, crude oil. The occasion for the investigation and the present report arises from remarks made by Mr. Krchma published in the 1938 proceedings of this conference. Mr. Krchma stated in effect that it was difficult to classify materials as cracked and uncracked on the basis of the Oliensis test and based his remarks on information contained in the Bureau of Mines Report of Investigation No. 3425 entitled ''Some Asphalts from Oregon Basin, Wyoming, Crude Oil,'' covering six high-vacuum distillations run in the laboratories of the Bureau's Petroleum Experiment Station at Laramie, Wyoming.

It appears that cracking had been suspected by the operators on two of these residuals due to condensation of some of the light distillates and a variation in the residues of these two materials. Mr. Oliensis undertook an investigation evidently in cooperation with the Bureau of Mines and others involved to weigh the evidence of cracking and checked the efficacy of the spot test in determining milder degrees of cracking than the standard test was ordinarily expected to measure.

In the case of one of these materials his conclusion is, ''There is found absolutely no evidence of cracking in Asphalt No. 5 in any of the physical or chemical test data available for that asphalt.'' This conclusion is verified in that there was no evidence of cracking in the spot test as performed in standard procedure or in extensions of this procedure which the author recommends for border line cases.

In the case of the other material, Asphalt No. 6, the author concludes from the physical and chemical data available in the Bureau of Mines report that the material is cracked to some degree. In this case the evidence of cracking was revealed by the spot test if the procedure was extended as recommended by the author.

The investigation includes the study of an additional residual asphalt, No. 7, which had been prepared subsequent to the release of the Bureau of Mines report referred to above and which had been subjected to higher temperatures than the preceding six materials and which did reveal positive reaction to the standard spot test. In the case of this material, while it included a treatment which was more severe only in the degree of heating, the spot test gave positive results in all cases.

The procedure followed by the author in testing these seven samples was closely in accord with that outlined in the appendix to his paper on ''A Further Study of the Heterogeneity of Asphalt—A Quantitative Method'' appearing in the 1936 Proceedings of the American Society for Testing Materials, Vol. 36, Pt. II, p. 506. He cites other references from which it appears that the usual procedure in making the spot test using filter paper rather than glass, after aging only 24 hours instead of a longer period, means sacrificing a certain degree of delicacy that was inherent in the original test. The shortened test procedure was set up as standard in the interest of greater speed and ruggedness and to make the test less subject to variations in the personal equation. It follows that a very slight degree of heterogeneity was purposely ignored in setting up the standard method so if the positive reaction did develop it would be accepted without question by all parties as evidence of clear cut heterogeneity.

The author submits the results of two series of spot tests on the two materials in question, one of which on Asphalt No. 5 shows no evidence of a spot after aging through successive periods up to 28 days. Asphalt No. 6, which was judged to be slightly cracked was negative up to 7 days but in successive periods of time showed positive evidence of heterogeneity. A second series of check tests was run on Asphalt No. 6 with identical results.

While the writer feels somewhat afield in this extended phase of asphalt technology, it would appear to him that Mr. Oliensis made an extremely careful investigation and presented a very good case for the spot test as a measure of various degrees of heterogeneity in the materials under discussion providing the procedure is extended as originally intended for these border line cases.

* * *

In his paper on ''The Measurement of Mechanical Stability of Bituminous Mixtures and its Correlation with Service Behavior'' Mr. Endersby of the Shell Development Company gives a description of the so-called triaxial compression test or stabilometer test.

The work reported is a part of a program of fundamental research which is being carried on in the Shell laboratories. The author notes that their problem is somewhat different from the problem of the construction engineer who has to deal with a limited number of sources of both asphalts and road aggregates. With a limited number of available materials it may be feasible to arrive at the best combinations from the standpoint of service behavior with relatively small amount of experience which can be readily correlated by some empirical tests. The producer, on the other hand, has such a wide range of materials with which his product may be combined that even in a lifetime program of trial and error he would not be able to completely evaluate his product. The producer must, therefore, rely on fundamental research which determines why certain combinations produce good results while others do not.

The method of approach favored by the author is then to investigate the fundamental properties of bituminous mixtures, it being considered that the important properties of the road surface are revealed by its resistance to deformation under applied load. The author believes that of the available mechanical tests the triaxial compression test involves the most logical set of forces to which the mixture should be subjected.

In discussing the principles of the triaxial test Mr. Endersby follows the classic theory which assumes that stability is a function of internal friction. He makes use of the Mohr circle method in establishing relations between the principal pressures measured in the test and the assumed source of resistance. The combination of cohesion and internal friction are also illustrated in terms of the Mohr circle. Mr. Endersby presents a summary of the data obtained on the frictional characteristics of aggregates and compares these data to the results obtained by some other investigators.

The range of pressures investigated was higher than ordinarily tested, vertical pressures used covering a range of 500, 750 and 1,000 lb. per sq.

in. In his summary of investigation the author lists as factors of internal resistance in bituminous mixtures, interlocking of particles, friction between particles, cohesion, and apparent cohesion. In reviewing the functions of mechanical tests it is noted that loading in practice results in compressive stress, tensile strength being of importance only under special conditions of temperature in loading.

For purposes of fundamental analytic research it is necessary to use tests in which these various resisting factors can be separated from one another. It is considered that the compressive test is fundamentally a shear test and cannot be analyzed except by an elaborate combination of conditions. The Hubbard-Field test also cannot be analyzed mathematically and should be classified as an empirical rather than an analytic test. As previously stated, it is felt that the stabilometer with the Mohr circle method of analysis gives results which can be more easily correlated, but this test is still a method of fundamental research and not yet adapted for routine testing. The correlations already discovered give promise but there is such a wide field yet to be covered between the semi-static test methods and conditions of practice that a very open mind should be maintained as to the practical value of all present tests.

* * *

A paper on "The Correlation of Laboratory Tests of Bituminous mixtures with Service Behavior" was prepared by Roland Vokac, formerly of the Barber Asphalt Corporation. The paper is a summary of extensive research made by the author and the analysis of samples from pavements in 18 representative cities in the eastern half of the United States. The results presented are in all cases the average results of a large number of individual cases.

The pavements are divided into three types from the standpoint of service behavior: Group A, pavements which have cracked; Group B, sound pavements which have neither cracked nor shoved; and Group C, pavements which have shoved. The results of these studies have been presented in a number of other publications but the summaries and conclusions are of particular value in relation to the subjects under discussion in this program.

In a study of mixture composition it is found that gradation and bitumen contents of Groups A and B are almost identical while Group C runs high in both mineral filler and bitumen. The important consideration appears to be that while Group C pavements may be the result of improper design of the mixture, the failure of Group A pavements by cracking must be accounted for in some other way.

The second set of data presented has to do with penetration data on the original and recovered bitumen. The average original penetrations were for Group A—49, Group B—51, and Group C—55. Again Groups A and B are practically the same with a nominal average penetration of 50 while the average of 55 for Group C indicates a more than negligible differential. The data on the penetration of the recovered bitumens is particularly interesting, the averages being Group A—18, Group B—29, and Group C—39. A correlation of these figures with the service behavior indicates that when the penetration of the recovered bitumen becomes less than 18 the pavements are definitely of the cracking type. Penetrations between 18 and 25 are prone to crack while those with penetrations over 25 are generally free from cracking. The drop in penetration is also significant and eliminates the effect of slightly different original penetrations. In this respect cracking pavements showed a penetration drop greater than 65, questionable pavements between 50 and 65, pavements without cracks a penetration drop less than 50 per cent. According to these figures specifications of the types promulgated by the States of Michigan and Virginia stipulating that a penetration of bitumen recovered from a mixture shall not be less than 50 per cent of the original penetration are justified.

The data collected on the ductility of recovered bitumen at 5 cm. per minute and 77° F. are as follows in terms of the average ductility: Group A—6; Group B—40; Group C—70. Interpreting these data in the same manner as the penetration, ductility of less than 4 cm. indicates pavements that are definitely of the cracking type, between 4 and 24 cm. pavements are prone to crack, and when ductility is over 24 cm. the pavements are generally free from cracking.

The data for the ring and ball melting point indicate that the average melting point of Group A pavements was 178, Group B 148, and Group C 142. These data may be interpreted to show that when the melting point of the recovered bitumen is over 160° F. the pavements are generally prone to cracking. When the melting point is less than 160° F. the pavements are generally of the noncracking type. Between 136 and 146 the pavements tend to shove while at melting points of less than 136 shoving would be almost certain in hot weather. The melting point data showing melting points in Group B and C of 148 and 142 are particularly interesting in the light of the general consideration that pavement temperatures may become as high as 140° F. during the summer months. This may result in the asphalt cement becoming soft enough, periodically, to flow or migrate sufficiently that the mixture may be self-healing.

The average age of the three groups of pavements are Group A—10, Group B—9, and Group C—14. It will be noted that Group C, the shoving type, has the greatest average age while Groups A and B are approximately alike, averaging 10 and 9 years respectively. If age were much of a factor in the hardening of the asphalt it might be expected that there would be a much greater difference in the ages of Groups A and B. Unless the bitumen in Group A is hardened much more rapidly it appears that this change in the characteristics of the asphalt more probably takes place during the mixing and laying operations.

Data is also given on the physical characteristics of the mixtures themselves, including density and voids of pavement samples and standard briquets made up from the paving mixtures, and mechanical tests consisting of the Hubbard-Field stability, the Skidmore shear, compressive strength, elastic limit, modulus of elasticity, and modulus of permanent deformation. The last three characteristics are obtained from the load-deformation diagram of a compression test on a standard briquet 2 inches in diameter by 1 inch in height.

Elastic limit is determined as the point of deviation of the load-deformation curve from a straight line. The modulus of elasticity is calculated from the slope of the straight-line load-deformation curve below the elastic limit, while the modulus of permanent deformation is calculated from the downward sloping straight-line portion of the load-deformation curve obtained after the compressive strength has been exceeded.

In the data presented on the density and per cent of voids, it is interesting to note that the density of the standard briquets molded from the paving mixtures were practically identical for all three groups, being approximately 2.21 while in the case of the pavement samples there is a noticeable decrease in density in the Group A pavements. Group A pavements showed an average density of 2.15 while Groups B and C have an average density identical with the standard briquets. The same data expressed in terms of voids in the mixture also differ in connection with the pavement samples in Group A which showed a void per cent of 7.1 as compared with 4.6 and 4 for Groups B and C respectively. It will be remembered that the mixture composition in aggregate gradation of Groups A and B are practically identical. It would then appear that the lower density and higher percentage of voids in Group A samples may have been a contributing factor in the cracking of these pavements, a fact which has been noted in the papers presented on that subject in the earlier part of this program.

The author has studied the results of the various mechanical tests from the standpoint of determining the accuracy with which these tests may be correlated with service behavior. The basis of this correlation is taken to be the accuracy with which poorly designed pavements could be identified by a quantitative measure of the physical properties involved.

Mr. Vokac concludes that no one of the three types of test given is capable of detecting all of the factors which may enter into a pavement failure but points out that the compressive strength test and the three derived coefficients, i. e., the elastic limit, modulus of elasticity, and modulus of permanent deformation provides a combination which comes closest to interpreting the possible behavior of a mixture in servcie.

* * *

Mr. Campbell of the New Mexico State Highway Department has prepared a paper on the very intriguing subject of ''Supplying Vitamins and Eliminating Bugs in Asphaltic Road Material Specifications.'' He notes that due to the rapid development in highway improvement and the demand for greater mileages of improved roads the use of asphaltic material very rapidly becames a business of large proportions. Because of the lack of knowledge of this particular type of construction such specifications as were available contained many imperfections and were woefully lacking in vitality. In addition there was no easily organized effort to adopt common specifications or even eliminate the multiplicity of products that were being produced for essentially the same purpose.

A great step forward was taken when the U. S. Bureau of Public Roads and the Asphalt Institute got together and evolved a common language and a simplified scheme of analysis and classified asphaltic road materials into a reasonable number of types and grades. In spite of this, good, bad and indifferent products continued to find their way into highway surfaces all over the county simply because the provisions aimed to secure quality were quite inadequate to cope with the situation. The solution of the problem seems to lie in evolving a set of test procedures which will dependably eliminate the inferior products. While it is admitted that this is easier said than done, Mr. Campbell suggests that a logical approach would have the following general objectives:

1. The use of only crude oils which were capable of producing good, live durable products.
2. Refining of these crudes without injury.
3. The elimination of excessive amounts of impurities and deleterious substances.

Recognized authorities have said in effect that asphaltic base crudes and semi-asphaltic base crudes in the order named produce the grades of asphaltic materials most desirable. Considerable information is available on the characteristics and properties of these crudes as well as those of the inferior paraffinic of nonasphaltic base methods and tests are available which serve well for identification. The author feels that Vitamin A would be the requirement that every crude which is proposed as a source of asphaltic road materials be sampled and submitted to the State Highway Materials Testing Laboratory for examination and approval before it is used either by itself or in combination with other crudes. Vitamin B is supplied by the specifications stating that the refined product shall not be burned, cracked, or otherwise injured in the refining process and backing that requirement up with anti-cracking guns that will do a good job of picking off overheated and cracked oil without eliminating the desirable uncracked materials. Vitamin C consists of a real specification eliminating deleterious materials of which it is felt that paraffin is the worst. The final vitamin injection would take the form of a statement in the specifications to the effect that border line materials will be definitely rejected rather than allowed to become controversial. This would be done on the grounds that there is no justification for permitting the use of doubtful materials.

With the specifications thus vitalized the author notes that there are still a number of bugs to be eliminated. There appear to be too many time consuming tests that have little real significance and serve no useful purpose. Another kind of bug are those which are highly involved or thickly populated with acute exactnesses which provide many points of vulnerability and rich grounds for controversy. Then there are certain conflicting tests which require the material to have some property which, although it may be desirable, cannot be obtained without sacrificing something else of equal value. There also are a number of tests which may be just exactly the thing for refinery control or well adapted to research which have no part in the specifications. These tests may be masterpieces of professional accomplishment in serving their original purpose yet they do not help a specification through no fault of their own and in fact may actually be detrimental.

In summarizing his remarks Mr. Campbel, in addition to favoring approval of the crudes to be used in manufacture and some degree of control over the processes themselves, suggests the following requirements which he believes would serve a useful purpose:

1. Water and sediment limitation (for purity).
2. Flash point (fire hazard, type identification, etc.)
3. Consistency (selected according to type of construction).
4. Paraffin and other deleterious substance limitation (for quality and purity).
5. Anti-cracking tests (for quality).
6. Solubility in CCl_4 (for identification and purity.)
7. Mechanical tests to identify type and grade of material.

In the discussion the author limited himself to liquid and semi-solid petroleum products and notes in conclusion that specifications covering aggregates could no doubt be put through a similar clinic with benefit.

* * *

Mr. Stackhouse, Construction Assistant of the Washington State Highway Department, prepared a paper which has been listed under the head of the ''Present Status of the Construction of Low-Cost Bituminous Surfaces.'' Such a discussion is properly a part of the program on research as investigators frequently have to check back to current practice to remind themselves of their ultimate objectives and regain their perspective. As noted by the author, methods of constructing low-cost bituminous surfaces are subject to a great degree of diversity of opinion and seemingly opposed ideas are being successfully employed in various parts of the United States. This fact has been certainly confusing to the laymen and there is even evidence among investigators that it is not always fully appreciated. Too often even those who should know better attempt to extend methods of procedure and design criteria that have been successful with one type of local material but which will positively not apply under an entirely different set of conditions.

Mr. Stackhouse restates an admonition which all of us have heard many times but which will certainly bear emphasis, that the factors that contribute most of the success or failure of a bituminous surface roadway are adequate drainage and a stable foundation. This is more noticeable in the low-cost bituminous mats than in the heavier pavements but the results of ignoring them are the same in the end. Those who are fortunate enough to have subgrades of natural sand and gravel need only consider adequate drainage. Those, however who have to build upon heavy clay or silt many times have perplexing and difficult problems to solve.

A few years ago the Washington Highway Department adopted a practice of laying a sub-base of crushed rock of from 9 to 16 inches in thickness over clay subgrades and felt that the problems would be solved. They found, however, that soft areas still developed due to water becoming concentrated in the sub-grade beneath the ballast. As the clay softened it squeezed up into the voids in the coarse stone ballast under vibration and traffic. This situation was corrected by the use of a sand or gravel cushion of sufficiently fine texture to prevent the squeezing action. The author notes that before any bituminous surfacing program is put into effect the engineers should be thoroughly satisfied that the road surface will continue to have a sufficient foundation and adequate drainage. In this connection it is noted that the construction of the impermeable mat may frequently change subgrade conditions by cutting off evaporation.

One interesting illustration was given of a section of highway in California across the Imperial Valley where the annual rainfall is no more than 2½ inches. While there is insufficient moisture to support vegetation yet in seven years after a bituminous surface was laid sufficient capillary moisture had accumulated under the mat to cause failure of the surface.

The paper presents some information on the construction features of several low-cost bituminous type surfaces consisting of:

1. Light bituminous surface treatment.
2. Non-skid seal treatments.
3. Bituminous surface treatment ''road mix'' type.

In the discussion of characteristics of the aggregate he makes note of the necessity of investigating its adhesion characteristics. From the standpoint of a research enthusiast the writer could not help but get a chuckle out of the following statement: ''The new material and especially quarried rock should be investigated for its hydrophilic or hdrophobic properties or in other words whether or not the material will allow the asphalt film on the surface to strip off the rock in the presence of water. I used to think these were good seventy-five cent words concocted by someone to confuse the profession and had no real meaning to us—but I know now there are some sources of rock in this state that do have hydophilic properties and are not good for bituminous types of construction.''

In conclusion, the author stresses the following points in the construction of low type bituminous surfaces:

1. Be sure the base or foundation is adequate and well drained. The bituminous mat will carry no more load than the foundation upon which it rests.
2. Use the best materials available consistent with the cost and use to which they are to be placed. Never use clay filler in any surfacing course to bind up the surface if you expect to oil the roadway.
3. Insist on competent and experienced inspection.

* * *

The paper on ''Recent Developments in Soil Stabilization'' has been prepared by Doctor Winterkorn who very kindly volunteered his services for this task. In his opinion soil stabilization is coming of age. The writer was mildly suspicious of the Doctor's simile of soil stabilization as a fair maiden just going to finishing school who is being forced to give some of her dearest aspirations as the awkwardness of youth is being stripped off. Perhaps her indiscretions can be excused because her youth has been spent in company with a rough school of practical engineers and she is now being finished off in a school of world-wise industry.

Doctor Winterkorn devotes some attention to the definition of stabilization and the classification of various types of stabilization. He selects as the most suitable:

1. Mechanical stabilization affected by manipulation and compaction of the soil.
2. Physical stabilization, which includes the preparation of graded soil mixtures by the addition of deficient fractions.
3. Chemical stabilization, which includes the addition of materials capable of providing stabilization through chemical reactions.
4. Physico-chemical stabilization, which covers the use of those admixtures whose reaction takes place through surface-chemical properties of mineral aggregate as in bituminous stabilization.

Mechanical stabilization as defined has been developed to a rather advanced stage in connection with compaction of earth fill dams and preparation of subgrades. Because every engineer is or should be intimately acquainted with the principles and practice of soil compaction, detailed treatment did not appear warranted in this review.

In connection with physical stabilization the author presents a summary of comprehensive investigation by Carpenter and Willis of the Bureau of Public Roads. This investigation includes the study of sand-clay and sand-clay-gravel materials for base construction. Two series of sand-clay mixes were investigated, in one of which the gradation was held constant and the plasticity index varied from 0 to 18. In the other, the plasticity index was held constant and the gradation was varied. Service behavior of the mixtures were studied under traffic by means of a circular track. Supplementary tests included compaction studies using the Proctor technique and compaction by vibration. Stability measurements using a Hubbard-Field apparatus were conducted on cores taken from the track, on molded specimens covering a range of moisture contents, and on molded specimens compacted at various compression loads and constant moisture content. Complete data on the various mixtures is presented in the paper in tabular form.

The results of these experiments can be summarized as follows:

1. The circular track can be used to great advantage in testing and classifying sand-clay mixtures.
2. For best service in the sand-clay system, both grading and plasticity must be controlled.
3. The failures which occured in Series 1 were caused by excessive plasticity while the failures occurring in Series 2 are traceable to improper grading.

4. A doubtful behavior of the sand-clay system is always characterized by a swing of the gradation curve toward finer materials.

5. The plasticity index of the –40 material should be below 6 for satisfactory performance.

6. Repeated experience has shown that if the material retained on the No. 10 sieve does not exceed 35 per cent the system retains essentially the properties of the soil mortar.

Based upon this series of tests a specification has been set up that is recommended for bases to be used under a bituminous surface treatment.

Further conclusions offered from this investigation are:

1. Control of grading, particularly by limiting the amount of material passing the No. 40 and No. 200 sieves is essential to insure satisfactory stability.

2. Excessive amounts of dust even if not producing a large plasticity index should be avoided as such material retards compaction and is conducive to failure under unfavorable moisture conditions.

3. Thorough compaction must be obtained if trouble is to be avoided, and the compaction obtained by vibration is a closer index than that obtained by the Proctor method.

4. Early difficulties in compacting mixtures possessing acceptable gradings and plastic indices cannot be taken as indicating poor quality in service. Rather the opposite seems to be the case.

5. Compaction of base courses having plastic properties should be completed to maximum density before the surface course is applied.

6. The Hubbard-Field stability measuring the rate of change in stability for a given change in moisture content appears to be highly significant as a means of predicting service behavior.

The study of sand-clay gravel materials for base course construction more or less parallels the study of the sand-clay mixtures and gave similar results. The following conclusions are, however, of additional interest:

1. The grading of the coarse fraction is of greatest importance if the mortar fraction decreases below 40 per cent, while the grading and plasticity index of the mortar fraction is a controlling consideration if this fraction exceeds 40 per cent.

In connection with physico-chemical stabilization or rather bituminous soil stabilization Doctor Winterkorn reviews several types of stabilization and procedures that have been offered for evaluating their physical properties. He notes that at the present time the main effort of the research engineer is directed to the urgent and important problem of establishing and standardizing laboratory test procedures to predict the service behavior of bituminous soil stabilization. The underlying principle for all test methods employed is simple and consists in the main of exposing the compacted bitumen-soil system to the effect of moisture with subsequent testing of the consistency of the system. Four different test methods which have been proposed show differences in the preparation of the soil samples in the methods employed to mold, compact, cure and test the specimens. The originator of any one of these methods has silently or openly criticised the methods of the others. The general discussion of this subject is still wide open and there is a large amount of work remaining to be done. Tabulations contained in the paper are of value in summarizing various types of stabilization and methods of testing but will not be included in this review.

It appears that the more important differences in the proposed tests lie in the methods of curing and water exposure of the specimens rather than in the type of stability test used. The proposed curing methods are:

1. Air drying for seven days.
2. Air drying with subsequent oven drying to constant weight at 140° F.
3. Air drying with subsequent oven drying to constant weight at 160° F.
4. No curing.

It is a well known fact that drying a moist bitumen-soil system may change irreversibly its future susceptibility to the effect of water. It is also known that in certain climates a thorough drying of the stabilized road will take place while under other conditions no water loss will occur at all. The author discusses the various aspects of these treatments and suggests combinations which may be used to approximate the field conditions to which the mixtures are subjected.

A report has recently been submitted on the physico-chemical factors of importance in bituminous soil stabilization by the Missouri State Highway Department. This report attempts to correlate the results of bituminous stabilization with the surface chemical properties of the soil and bituminous materials used. General results of this study indicate that the service behavior of these mixtures depends to a large extent on the chemical character of the finer soil constituents which may be expressed by the silica-sesquioxide ratio of the clay and colloid particles and by the amount and kind of exchange ions. From the standpoint of the efficiency of the bituminous materials they can be divided into three groups:

1. Best—positive asphaltic cut-backs and high temperature tars.
2. Intermediate—standard negative bituminous materials.
3. Poor—low temperature tars.

It is also noted that a study of the behavior of sodium and potassium saturated soils seems to explain the failures of bituminous treatments in alkali soils and indicates that furfural type activators can be strongly recommended for such soils.

Under chemical stabilization the most important type listed is that of hardening of soils by the use of Portland cement. The development of the basic principles of this type of soil stabilization has been rather widely publicized and need only be summarized very briefly. Tests used to evaluate service behavior are logical and seem rather severe. The procedure includes the compaction of the soil by use of the Proctor technique and the measurement of resistance to deterioration under wetting and drying and freezing and thawing. The results of the Portland Cement Association can be correlated to some extent with the routine soil tests as indicated by Sheets and Catton that effective hardening can be achieved with soils having a liquid limit below 50, plasticity index below 25, clay content below 35, and a percentage of solids at maximum density equal to 60 or greater provided further that the soil possesses a regular moisture density curve.

Doctor Winterkorn states that since all five of these properties of natural soil can be changed by ionic treatment it appears that some form of chemical treatment may be combined with Portland cement with benefit.

Experiments carried out by the Missouri State Highway Department on one type of lateritic soil revealed a great influence by such pretreatment and still greater effects can be expected with podzolic soils which are more conductive to ionic treatment. This brings up the old question of the correlation of the surface chemical properties of soil with their susceptibility to stabilization. For this reason it can be expected that the chemical classification of soils will also be of great value for the further development of soil hardening. The large amount of available information on the chemistry of American soils promises easy advances in this direction.

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Albuquerque, New Mexico.

Mr. Housel: Gentlemen, after listening this morning to Professor Lang's paper, I couldn't help but feel that both the research men and the producers of bituminous materials were put on the spot, so to speak. Inasmuch as the research men have the first chance here, it might be possible for us to so maneuver things as to leave the producers of bituminous materials facing the firing line alone, but I don't think that would be a wise thing to do.

A remark I heard in a conversation this morning seems to me to fit the situation rather well, and that is, "that we should hang together or probably we will both hang separately." With that in mind it seems to me that both the research men and the producers of asphaltic materials, as well as construction engineers, should all face the challenge that Professor Lang presented in his paper and at least try to approach the rather optimistic forecast he set up for the next eleven years.

In presenting the review, I have thrown myself on the mercies of the court and I admit frankly that I am not an asphalt paving technologist. During the last few years there has been a considerable interest in soil mechanics and, as many of you know, soil mechanics in bituminous construction is the route by which I have become more or less interested in your particular field.

I am going to pass the buck today to the members of the Conference Board. I will simply act as telephone operator in making connections between the audience and the Conference Board.

In addition to the papers which were submitted and included in the review, there have been a number of supplementary papers which have been turned over to the chairman just prior to the meeting. This situation has arisen because of the fact that we had so little time this year to prepare papers, and that is true of all the sections—that a number of the men were unduly hurried, and they have some supplementary information to submit at this particular time. Probably the best way to do that is to present these papers as a part of the regular discussion.

I have a paper by Dr. Tucker of the Standard Oil Company. Dr. Tucker's paper supplements the one presented by Mr. Juhlin and gives some additional data on the factors in the hardening of hot mix asphaltic materials. It includes work that has been done by Dr. Tucker and the Standard Oil Research Laboratory. In a rather hasty review of it, it appeared to me there were three points that deserved to be presented, which are more or less in corroboration of the results of the Michigan work in the hardening of asphalt. The important things that are indicated are that different asphalts have entirely different characteristics with respect to hardening, as brought out in some respects in the previous paper by Mr. Schaub. In the first place, the type of asphalt, depending upon the source, is an important factor, and second, the treatment of only particular crude may cause the final material to be of an entirely different type with respect to age hardening characteristics. A third point which Dr. Tucker has brought out is the fact that, because the viscosity of the various asphaltic materials is quite different, there is an opportunity to obtain the required consistency for mixing in the case of some materials at lower temperatures than in the case of other materials, which may avoid to some extent the hardening effects that take place during mixing and placing.

I have also a paper, "Cracked vs. Uncracked Road Oils," by Mr. Clark, Chief Chemist of the New Mexico State Highway Department, which was submitted to supplement Mr. Campbell's paper. After the recess, or at the beginning of the discussion period, I am going to ask Mr. Clark to summarize that paper and present the main points to the Conference.

That, gentlemen, completes the review, and I believe we will follow the program in recessing now for a few minutes.

In opening the discussion, it seems desirable to keep it confined to certain aspects until they are completed, I think it will be well for us to consider first, under bituminous materials, the first three papers which have to do with the general subject of the hardening of bituminous materials. Those three papers are now open for discussion.

As I stated at the beginning, I am going to pass questions that come up on the floor to members of the Conference Board who prepared the various papers, and perhaps will call on various individuals to supplement the discussion from time to time. However, this is your meeting, and I first want to give everyone on the floor an opportunity to participate.

H. G. Nevitt: Missouri

I would just like to raise a point in connection with the hardening of asphalt, whether we are so much concerned with the percentage drop in the penetration as with the final penetration. In other words, there is the possibility that if we get a certain final penetration or a final penetration within a certain range, it is immaterial what penetration we start with in the mixing and laying operation. I don't take any position. I merely would like to raise that point for some comment by the Conference Board.

Mr. Housel:

You have all heard Mr. Nevitt's question. Is there anyone from the floor who would like to comment on that? If not, I will refer that question first to Mr. Juhlin. Do you have anything to say on this, Mr. Juhlin?

H. A. Juhlin:
Illinois

From the information we seem to have, I don't think, Mr. Nevitt, we should really be concerned as much about the percentage drop in penetration. At the present time, thirty has been the arbitrary figure for the eastern part of the country but, as pointed out in the papers, there have been several instances in Arizona where it went down to as low as fifteen. From the information we have been able to obtain, it would seem we should not go according to percentage, but by actual final penetration based on the arbitrary temperature at the present time of 77.

Mr. Housel:

Mr. Schaub, would you have anything to say on that?

J. G. Schaub:
Michigan

As Mr. Phelps has pointed out, engineers are following the results of these various researches that are being made and using softer asphalts, lower temperatures and other favorable practices in handling asphalt in order to prevent a decided drop in penetration. Besides that, we must recognize that there are differences in asphalts, as was pointed out in Mr. Juhlin's paper and by Mr. Campbell. There are certain asphalts from certain sources, and due to certain methods of manufacture which, in spite of favorable handling, will have greater drops than are safe to have in paving practice, and it was with that in mind that Michigan adopted its rather stringent requirements of penalizing the contractor for abnormal drops or excessive drops in penetration and ductility. That is a point, I think, that the producers should bear in mind. While the engineers are endeavoring to use asphalts in the most efficient manner to prevent those drops, there are certain sources and methods of manufacture that still cause excessive drops which we must guard against.

H. G. Nevitt:
Missouri

Isn't is true, though, that you could take care of those questionable ones by allowing a greater initial penetration? In other words, is it not a debatable point that, provided this hardening does not continue afterward which is a matter to be determined, you could start with a softer asphalt from these sources you talk about, also provided you had other data to indicate they were not unsatisfactory in other ways, and end up with a satisfactory pavement from the hardening standpoint.

In other words, might you not write your specifications on the basis of a final penetration with a permissible initial penetration, or perhaps any penetration at the start, as long as immediately after laying you were in an acceptable range?

J. G. Schaub: Michigan

That is true in a general way, but is it good engineering to adopt such high penetrations to try to protect the few questionable and inferior asphalts that we run across occasionally? That is the point we raise.

H. G. Nevitt: Missouri

I made the point that the asphalt, judged by whatever standards you want, are satisfactory otherwise, and I made the qualification that it must not continue to harden with age beyond the usual rate. I believe you will find asphalts which have a very satisfactory experience over the world will come under that classification.

Mr. Housel:

I am a little bit hesitant to enter into this discussion of asphalt technology, but isn't it true, Mr. Nevitt, that an asphalt which does show a susceptibility to hardening during mixing and placing, even though you might start out high enough, gives an indication of probable continued hardening for the same reasons during a period of life in the pavement, unless you can adopt additional precautions of making a tight mix, seal coating, or something of that sort?

H. G. Nevitt: Missouri

You notice I am very careful in not taking any position. The reason I avoid it is that we don't have any data. We don't have sufficient evidence to show that some asphalts which show a rather rapid initial hardening, then fail to continue to harden, resemble these other asphalts which have a low initial hardening. It so happened that asphalts I have in mind, which may fit into that classification, are not usual around our part of the country. I cannot have the familiarity with them and I don't believe they have met the Michigan specifications, so we are talking about something that we don't have data on, but some things we have read and the little work we have done has led us to suspect there may be no relation or no great relation between initial hardening in the process of mixing and laying, and continuing hardening after laying. If actual surveys or information shows that is true, it would seem very unwise to set a specification which is concerned with the rate of hardening initially, and more wise to make the specification take care of the penetration after the laying, with whatever safeguards are required to make sure that that asphalt does not continue to harden at a rate beyond the normal.

J. G. Schaub: Michigan

Of course, we haven't sufficient data as yet to be able to make any real dogmatic statements in this regard, but let me cite you an example that we had a few years back on a sheet asphalt pavement in a large city of our state. It took nine cars of asphalt. Two cars failed to meet our recovered ductility and penetration test. The producer was quite concerned, having stated that the whole nine cars came from the same source and were manufactured in the same way. It led to considerable controversy and involved various meetings with the officials of the company and their refinery superintendent. The final analysis was that the two cars in question came from a different crude than the other materials, and the drop in penetration and in ductility was so markedly different in those two that there certainly must have been something in the source of the material that caused it.

V. A. Endersby: California — We have some data on that. I haven't anything here that I can present quantitatively in terms of actual figures, but European experience with quite a large number of asphalts on different projects corroborates the thirty penetration point as being dangerous.

In regard to differential hardening between different asphalts, we have found in laboratory and semi-field experiments that an asphalt which hardens rapidly in the mixer does not necessarily harden proportionately as rapidly afterwards. That is, some appear to be more susceptible to hardening by heating than they are to hardening by time exposure, and vice versa. We found, in addition to temperatures, during mixing, which have been pointed out in previous papers, that the void contact in the aggregate is probably the most important factor. We exposed mixtures with Ottawa sand which, as you know, has about the highest possible void contact, finding that a few weeks' exposure on such a material as that will sometimes produce as much hardening as several years will in a normally well graded thoroughly dense mix.

Also, observations in road practice in Europe show that if voids are as low as two and one-half per cent, which is sometimes the case in sheet asphalt in that region, no appreciable time hardening takes place after the material is placed.

I think the point of importance involved there is that when we speak of penalizing asphalts by checking up the penetration after being placed, it is not easy to say offhand what it is or who it is we are penalizing. Asphalts are variable in that respect. Aggregates are still more variable, heating and mixing are variable, and there is a certain amount of variation between engineers, also.

Mr. Housel: Are there any other comments? It seems to me, in the discussion as I have heard it—I am probably going to get into trouble before this meeting is over—that we are still avoiding the real issue in Mr. Nevitt's question.

Mr. Endersby has brought out that if the voids in the mixture are kept low your asphalt does not harden. Our results on the sheet asphalt pavements in Michigan show the same thing. There is a very marked change in the rate of hardening after the pavement has been laid, but that is due to mixture design. The asphalt is still susceptible. Isn't it a safeguard to try to eliminate those susceptible asphalts even though you may offset them by better mixture design and by seal coating, and so on? Isn't the penetration drop a safeguard that the engineer will still want to retain?

Then there is another point that comes up in that connection. Heating during mixing merely accelerates the oxidation. The hardening after the pavement is done is still oxidation, is it not; that is, ruling out thixotropy, which is a temporary change which may not be nearly as serious as the oxidation?

So, in answer to Mr. Nevitt's question, it still seems to me that the susceptibility, which is exposed by a big drop during penetration, is a dangerous thing for the pavement after it has been laid and is definitely related to that drop.

H. G. Nevitt: Missouri

It depends a lot on what is causing this drop. Oxidation, of course, is being blamed, and there is no question that oxidation may be a large factor. However, as Hveem likes to emphasize in discussion—I wish he were here—you have to establish correlation with practice before you are quite sure your theory describes the facts. It may be, though, that oxidation is not the only factor. It may be that evaporation of very small amounts of asphalt—the so-called light fractions in the asphalt may be another source of age hardening; that is, of hardening during mixing and laying. If so, exposure to the air will obviously result in the opportunity for these fractions to distill off, particularly at high temperatures. In other words, the high temperature, the presence of thin films, the exposure of the films to the air all are conducive to additional or increased evaporation of these very small light fractions. If that is the case and you had a small proportion of these light fractions left in the material due to its nature, or perhaps due to the refining, and then you eliminated these, you would then have practically no further drop in penetration due to them, and would have a perhaps normal asphalt. So that is one factor.

Now there is another factor. If you have the same amount of distillate, the effect of that on the penetration is quite variable. In other words, there are certain asphalts in which the removal of one-tenth of one per cent of the product causes a decided penetration drop. There are other asphalts in which the removal of the same quantity will cause a very small penetration drop. So I would agree, if oxidation was the factor and proven the only factor, the rapid drop at the start might indicate the possibility of a rapid drop later provided, as you point out, the opportunity for such oxidation occurs. On the other hand, if oxidation is not the factor and evaporation is a large reason for the penetration drop, you would have a different situation because you might remove a small amount of light volatile and from then on have practically no further penetration drop.

While the whole thing is not settled, because we simply do not have the laboratory information, I think it is quite possible that we will find out that the initial penetration drop is not a measure of the ultimate tendency to harden in the pavement, and under those conditions the securing of a proper penetration after laying would be the only requirement for a satisfactory pavement from the standpoint of hardening.

Mr. Housel: Are there any other comments or questions? If not, we will go on to the next subject, which is the paper on quantitative methods of measuring adhesion of bituminous materials, by Dr. Winterkorn.

Are there any questions or comments on the subject of stripping?

Mr. Endersby, I believe you have something in your review of that paper that may be of some interest.

V. A. Endersby: California.

There is one point in particular which is rather interesting in that connection. Some years ago, in connection with the construction of the San Francisco Bay Bridge, where aggregates for the piers were batched out of the plant on shore and then hauled out on barges for work, we had extreme difficulty in handling the water-cement ratio. Sometimes the materials would turn out to be too dry to work, with the prescribed amount of added water. The next time they would be altogether too wet. That, of course, caused trouble between the engineers out in the Bay and those on shore, each blaming the other.

It occurred to me that part of that trouble might be due to humidity, although it didn't seem probable the material could absorb that much water under varying conditions of the air. I found that it did, by means of a hydrometer. The difference appeared to be due to the fact that this material was stored in bins over night in the Bay atmosphere, which sometimes was so humid that great amounts of water absorption took place. I thought that point would be of some interest and value in connection with what Dr. Winterkorn has to say with regard to humidity. It would seem it is not a matter of whether actually raining or not, but of whether there is a high percentage of humidity in the air; that is, in handling road mixes and other cold materials at relatively low temperatures.

Mr. Housel: Dr. Winterkorn, do you have anything to add? As I understand it, you controlled humidity in your experiments, didn't you?

Dr. H. F. Winterkorn: Missouri

Yes, sir. In the course of our experiments on adhesiveness of asphalts to mineral aggregate we sometimes encountered difficulties in duplicating stripping data if the aggregate was coated on different days. Checking every possible variation of conditions, only the weather was left, and it was found that aggregate coated in a moist atmosphere, although the surface of the aggregate appeared dry, showed less stripping resistance than the same aggregate coated in a drier atmosphere.

Since that time, before coating we allow the aggregate to attain equilibrium with a definite water vapor pressure, by placing it for at least 24 hours in a dessicator containing an aqueous solution of sulphuric acid possessing a definite vapor pressure. The vapor pressure chosen in the experiment should be similar to that obtaining on the job. The following table gives the water vapor pressures for aqueous sulfuric solutions at 30° C.

Concentration% H_2SO_4	Vapor Press. mm. Mercury
95	
90	.0117
85	.0585
80	.183
75	.580
70	1.44
65	3.09
60	5.41
55	8.29
50	11.3
45	14.7
40	18.0
35	21.2
30	23.8
25	26.0
20	27.8
10	30.0

I think I should stop now. I have had my say about adhesiveness in so many meetings that I am getting a little bit tired of talking and repeating more or less the same things. There exists a curious knack in my mentality; the more often I hear a theory stated, even if I state it myself, the more suspicions of its weakness arise in my mind. I am sure that you would not want to disturb the serenity of my mind today.

R. M. Schwegler:
Oregon

I would like to bother Dr. Winterkorn a little more. We have found in the West that asphalts from different fields give different stripping results with a given aggregate. There seems to be quite a wide variation between different asphalts when tested on the same aggregate. Can you give any explanation?

Dr. H.F. Winterkorn:
Missouri

Of course, asphalts vary in their adhesive nature, although I think the aggregate and primarily the design of the mix have most to do with the final behavior of the road. If I remember rightly, in one of the papers Mr. Zapata stated that for the same type of bitumens the adhesiveness increases with increasing specific gravity up to one. This coincides with the results of some work we have been doing on bituminous soil stabilization where the problem is the same. In bituminous soil stabilization you cannot actually strip the bitumen off the soil, but you can determine the amount of water the soil takes in after it has been treated with bitumen, or the stability of the specimens after exposure to water.

We have been using quite a large number of different bituminous materials and we have found that, in general, the adhesiveness of the asphalts can be correlated with their content of asphaltenes. However, we found also that the California asphalt fell out of line. It showed excellent adhesiveness and not much asphaltene content. undoubtedly, there exists a close connection between the chemical composition of an asphalt and its adhesiveness; but it is as yet impossible to state it in a quantitative way.

R. M. Schwegler:
Oregon

We have found some remarkable differences in testing aggregates for stripping properties which involves both the natural fines occurring in the pit and the manufactured fines. We found on one aggregate that tests for stripping with asphalt from one field showed good adhesion on the natural fines and failure on the manufactured fines. The same aggregate was tested with asphalt from a different field and the results were reversed; the natural fines failed on the stripping test and the manufactured fines showed good adhesion.

Dr. H. F. Winterkorn:
Missouri

The problem of adhesiveness is complicated enough if you have one and the same mineral aggregate with variations in moisture and air, and all kinds of other substances absorbed on the surface. If you have two types of minerals, i. e., fines and coarse material of different origin, you introduce a new set of factors, rendering the behavior of the system scientifically unpredictable. We have to test.

When I first came into this field years ago I was quite an enthusiastic physico-chemist and thought all the outstanding problems could be solved in no time. In those days I did not see much reason for the existence of a lot of testing equipment, and testing engineers; however, the older I get the more I am inclined to think that the testing people fulfill a function which is very, very valuable; in fact, more valuable than a lot of things we physico-chemists do. Physico-chemists are sometimes led astray, (and are quite often led astray) because they simplify things too much. As in this adhesiveness business, I think a lot of wrong has been done by people who simplified the problems involved into, say, angles of contact, and other phenomena of this sort, and who emphasized surface and interfacial tensions, at the wrong time and in the wrong place. It is true physico-chemists have found very nice correlations between surface tensions and adhesive properties, which in theory are beautiful. However, what is true for certain types of molecules with nice symmetrical properties is not true any more if you deal with molecules which possess different properties on different places of their bodies. The molecules of which bitumen is composed are not obliging enough to be of the nice kind which the physical chemist can easily handle.

The testing man has the right and the duty to test anything which a theoretical fellow works out. The only solid basis for us to stand on is actual experience. Our theory should only serve to interpolate between our experience. We cannot assume anything to be true without having tested it and actually found out. As a fellow who, you might say, started more or less from a theoretical field, I think I have the duty really to emphasize this point: Don't trust us too much.

Mr. Housel: I was very much intrigued in Dr. Winterkorn's paper by the mention that was made of the importance of adsorbed air. I understand Mr. Powers has been investigating that angle. Is Mr. Powers here by any chance?

H. G. Nevitt: Missouri — He can't attend this year.

Mr. Housel: That is too bad, because that sounded like a very interesting angle, and probably one to which we have paid little attention, that is, the effect of adsorbed air films as well as adsorbed moisture films. I don't know whether you want to say any more about that or not; do you, Dr. Winterkorn?

Dr. H. F. Winterkorn: Missouri — It is rather difficult for me to take a stand. I don't know whether Mr. Powers wanted me to talk too much about it. He showed me some data and asked my opinion about some of the work. I thought it interesting enough and of enough value that it should at least be brought out to the public. He may feel he has enough data to give a paper. It cannot be foreseen yet what will come out of this work, but I was intrigued by his method of measuring the amount of oxygen adsorbed on the aggregates. Since the absorbed material doesn't weigh very much, Mr. Powers puts his aggregates in an alkaline pyrogallol solution and can determine by the color change how much oxygen is absorbed. I thought it was just a beautiful method if you can understand the feelings of a chemist.

If I remember rightly, it seems to me that materials which hold a lot of oxygen absorbed show quite some stripping, especially this oxidation type of stripping.

This work seems to touch on a very important phase of the adhesiveness problem which has been neglected up to date. Often we see some material, which has failed on the road, which isn't clean. It just has a thin, oily film around it. Here the stripping is apparently not a matter of differential affinity for water. Somehow the film around the aggregate has been changed in a way it cannot hold the extra thickness of bitumen along with it.

Mr. Housel: Are there any other comments or questions on this question of adhesion?

H. G. Nevitt: Missouri

I might just make a comment. I also know about Powers' work, and I certainly hope he gets to publish it because it is a fine job in my opinion.

I had one job about ten years ago on which, until Powers' work came out, I couldn't explain some things that happened, but now his air interfacial tension idea explains it. It looks to me that he is certainly on the track of something.

Mr. Housel: If there is no further discussion on that subject, we can pass on to the next. I think the next two papers might well have been included together in our discussion, ''The Interpretation of Current Tests on Bituminous Materials,'' by Mr. Zapata, and ''The Efficacy of the Oliensis Spot Test on Slightly Cracked Asphalts.''

There is one thing in Mr. Zapata's paper, a question that came up in my mind that I would like to have some comment on, possibly from producers as well as the Conference Board: Just to what extent do you feel it is possible to take an asphalt from a certain source and by the use of solvents split it up into three different components and then combine those components in synthetic mixtures and have any assurance that the material is at all representative of the original product, and what weight would you place upon the fact that they run check tests on the original materials and synthetic materials of the same composition?

Do any of you asphalt technologists have any thoughts on that question?

Dr. H. F. Winterkorn: Missouri

I should like to know whether Mr. Zapata gives an exact description of how he separated, and what he did to exclude air, for instance in his separations.

I think we all know that if we separate asphaltenes from asphalt they absorb a lot of oxygen. Of course, if you don't take precautions to prevent the absorption of oxygen, working either in inert atmosphere or in a vacuum, then you are not sure what you will get. It is very important in all work of this type that the actual procedure is really described. We sometimes find, not only in work done by highway departments, but also in work done by industries, that in a very important piece of literature the actual method of how separations were made is not given, I think we cannot afford to believe any data when the actual decription of the apparatus is not given. I think that goes for the users, as well as the producers. I have often heard the work of users criticized by representatives of the industry, who say, ''You have to do this,'' and ''You have to do that,'' but their own publications do not always give the essential data.

I want to make an appeal that all the essential data be given in any paper which is to be of scientific value.

Mr. Housel:

In answer to the question you have raised, Dr. Winterkorn, in the abstract I left out a good deal, and I am not sufficiently skilled in this particular field to know how adequate his explanation is. There is one paragraph which gives some detail on the method of combining these various fractions.

''The separation was based on the fractionization of the products according to molecular weights by means of a series of solvents of different internal pressures. Hexane was used to precipitate high molecular weight fractions, the asphaltenes, and liquid propane was employed for separating resins from a lower molecular weight fraction, the oily constituents. These fractions do not represent definite hydro-carbons, but are rather arbitrary separations defined by the method of separation and by the solvent.''

That, so far as I know, is as close as he comes to giving an actual description and, frankly, that doesn't mean too much to me.

I also should point out that the detailed results of this comprehensive investigation are given in the University of Wisconsin Bulletin, which is published jointly by Professor Hougen and Mr. Zapata.

Dr. H. F. Winterkorn:
Missouri

The point I would like to know is whether he worked in inert atmosphere or in a vacuum after he separated those fractions.

H. G. Nevitt:
Missouri

As I recall his method, he kept his asphaltenes covered by the solvent until he was ready to use them. I do not recall whether he mixed the materials and then removed the solvents so they were never exposed to air, or whether he removed them and then immediately mixed them in. I think he watched that point you mentioned.

Dr. H. F. Winterkorn:
Missouri

My general remarks were not about Mr. Zapata's work. I just wanted to stress the importance of getting all those data, so that when you read papers you actually will know what has happened. The mechanism of obtaining data, and the data themselves are very often much more valuable than the conclusions and opinions of the writers. Your data will always have value, while your conclusions may be changed by new knowledge which becomes available.

Mr. Housel:

Are there any comments on the other paper, "The Efficacy of the Oliensis Spot Test on Slightly Cracked Asphalts"?

I was hopeful Mr. Krchma would be here, as his remarks at the last year's meeting evidently inspired the investigation in the paper that was submitted which, so far as I could determine, indicated a rather delicate criterion by the extension of the spot test beyond the standard which has now been set up. I am wondering if any of you have had any experience along that line, or have any comments to make.

You might note that Mr. Zapata in his paper indicated that the spot test did not serve very satisfactorily in identifying the degree of cracking or heterogeneity in some materials. His statement in that connection is: "Unquestionably the Oliensis test has filled a need in asphalt technology. The results that are herein reported, however, indicate that it is possible to obtain positive spot tests on materials that have not been overheated, cracked, blown, or which do not have wax, sludge, or incompatible fractions." Then he goes on to cite a number of compositions which they made upon which they obtained positive spot tests, and yet there was no indication of cracking in the usual sense of the word. That is a little bit aside from the question of whether various degrees of cracking can be identified by the spot test, but it does throw some light on their experience with that particular test.

Do you have anything on that, Mr. Nevitt?

H. G. Nevitt: Missouri — I think I know what Krchma would say. He would make two comments. First of all, he was talking about the spot test as actually run; in other words, as a practical test. I think we will all agree that a 7-day, a 24-day, or whatever period Dr. Oliensis finally held his material to prove it is cracked, is not in the realm of a practical test for specification work ordinarily.

I think the other point he would make is that with Oregon Basin crude, quite possibly the Oliensis test will show rather a delicate degree of cracking due to the nature of Oregon Basin crude. It is quite sensitive to that indication. However, there are other crudes in which I do not believe that is the case. I think they can be cracked to quite a little extent before the Oliensis spot test will show it, if you are willing to accept some other criterion of cracking, such as an arbitrary time and arbitrary temperature, and say that anything held at that temperature and for some time is cracked. You may find that the Oliensis spot test will not indicate that, unless you hold it so long that the degree of cracking becomes very pronounced.

R. M. Schwegler: Oregon — I would like to mention the modified Oliensis spot test which has been developed in California. It uses a different solvent, xylene and a normal heptane. It calls for xylene 35%, and 65% normal heptane. I wonder if anyone here would care to discuss that test.

Mr. Housel: Is there anyone in the audience who has any knowledge of that test, or can answer Mr. Schwegler's question? How about the Board?

R. G. Clark: New Mexico — I might say, in connection with that modified Oliensis test, that we had a little experience with that down in New Mexico, or at least we tried it just a few weeks ago and, as far as we can tell, that is probably a pretty fair modification of the Oliensis test if you are just looking for cracked oil. But if you use the Oliensis test for any other purpose, the modified Oliensis is too wide in its interpretation. In other words, it will catch only the stuff which would have been positive under the normal Oliensis test.

Mr. Housel: Does that help some, Mr. Schwegler?

R. M. Schwegler: Oregon — I think Mr. Johnson of Montana can help us out a little.

Guy Johnson: Montana — If you take this modified Oliensis test and vary the percentages of these two reagents, you can rather roughly define the percentage of cracking. As you vary the percentages of heptane and xylene, I think you can make it equally as sensitive as the original Oliensis test. We haven't gone far enough with it to be positive on this point, but I believe further work is indicated along that line.

J. E. Buchanan: Idaho — I know something about the development of this modified Oliensis test. Although I am not sure I am up to date, because I understand some further work has been done, I can give some background concerning its original development.

It seems that some crudes as they come from the ground show a positive test or show some heterogeneity even before they ever see any kind of a still or any kind of a refining process. It is, of course, generally recognized that asphalts differ in some degrees. So this test was developed after a series of investigations by several laboratories, including the California Division of Materials and Research laboratory, and producers' laboratories in that area. The 35% Mr. Schwegler mentioned was the figure tentatively adopted for initial trial as the dividing line between materials which would be accepted and materials which would be rejected. That 35% is subject to future revision. (An asphalt which is ''negative'' when tested with a 35% solvent may actually contain a very small proportion of unsaturates, but the amount is at present considered to be inconsequential.)

Just as Mr. Johnson of Montana said, the test can be quite sensitive by varying the percentage of normal heptane and xylene. I have understood—it is said, at least—that the test is sensitive enough for refinery control, which is some indication of its possibilities. The per cent status of the test is based on the fact that it is generally believed that ''cracked'' materials are, on the average, somewhat inferior to materials which are uncracked. There is yet no conclusive evidence to prove that belief. I will simply say that is the general attitude.

This modified Oliensis test was developed as a reasonable working compromise to exclude asphaltic materials injured during refining. I may not have made it entirely clear, but this test is the result of a development which has been under way I think for nearly two years and, at this time, is being used for acceptance and rejection on the basis of negative or positive indication when tested with a 35% solvent.

Dr. H. F. Winterkorn: Missouri — I should like to hear whether Mr. Benson has some more data, or anything about the correlation of his test with that of Mr. Oliensis. I think Mr. Benson has made a very important contribution in the matter of heterogeneity from a different angle, and I am very much interested to see how those things can be brought together, or whether they can be brought together.

J. R. Benson: Kansas

I have been trying to keep out of this. I am to present a paper tomorrow in which I will endeavor to discuss the Oliensis test a little bit more. I don't believe I have anything particular to give at the present moment regarding the Oliensis test and some of the other work we have been doing in Kansas.

With regard to some of the comments on California's use of a normal heptane and xylene mixture for running the Oliensis test, it appears to me what you have really done there is to have shoved downward toward the positive side the material which you call negative. In other words, materials which might be rejected, as was stated, under the straight Oliensis test might be accepted under the modified test. Of course, that brings in the borderline cases of the Oliensis materials which are very controversial to begin with, and it would seem in my opinion that a great deal of work should be done and will have to be done, in fact, before the value of the straight Oliensis test or any modification of that test is definitely proved.

Those are about all the comments I have to give at this time.

Mr. Housel:

We appear to have pretty well exhausted the opportunities for discussion of those papers. Our time is getting a little short, so I am going to pass on to the next section on bituminous mixtures.

The first paper is on "The Measurement of Mechanical Stability" by Mr. Endersby, which in brief is a discussion of the stabilometer test and its advantages in measuring the physical properties of bituminous mixtures.

Are there any qustions from the floor on this paper on the general subject of stability as measured by the stabilometer test? I understand there are several men at the Conference who are interested in building stabilometers and perhaps doing some work on that. Now would be an excellent opportunity for them to ask at least some of the questions which they may have in their minds.

Mr. Endersby has prepared some supplementary information which will be included in his paper in the proceedings, which he was unable to have ready in time for the review of the papers.

Mr. Endersby, would you like to touch on the high points of that additional material?

V. A. Endersby: California

This material in the paper and also in the supplementary remarks deals much more with aggregates than it does with mixtures. The reason for that is that we began, as seemed logical to us, by working on bituminous mixtures, but found that the peculiarities of the aggregate were such, and had such complex results, that we had to go more deeply into the action of the aggregates themselves.

Mixing asphalt with an aggregate does not eliminate its frictional characteristics, its interlock and expansion effects, and the rest of it, but only modifies them. We are now endeavoring to get a systematic relationship by which one can test an aggregate without first mixing it with asphalt, and thus foretell what its characteristics are going to be in the mix. We have made a certain amount of progress.

One of the interesting points arises from the pressure ranges which are used. That is to say, in testing the apparent internal friction of an aggregate the coefficient of friction depends partly on the load being less with greater loads. The differences are considerable, and they change most rapidly, within the range of pressures occurring under traffic. That is to say, an ordinary balloon tire will put on a pressure under a standing load of about 40 pounds per square inch. Under impact it can be expected to go up to perhaps twice that. A solid tire or other special means of load application will apply perhaps 200 or 300 pounds per square inch. The Amsterdam laboratory of the B.P.M. uses a limit of 400 pounds, which is also what the State of California uses. Results tend to be more consistent with higher pressures, up to 1,000 pounds, or so. But then one gets out of the range of practical application.

The reason that friction is considered to be lower with a higher load is generally assumed to be greater abrasion of the material under the higher loads. We tried to find out just what the meaning of that effect was, and to do so we applied a heavy load, 600 pounds or so, obtained the coefficient of friction under that load, then dropped back on the load while continuing the deformation on the same sample, finding the curve to be about the same. When we got back to 25 pounds per square inch, we still had a much higher coefficient of friction than we had with the heavy load.

The point involved is that if there is sliding friction in the material, it is to be assumed it will be smoothed down as much as possible by the higher load to begin with; therefore, when you lower the load you shouldn't get a higher coefficient of friction. The fact that we did get the same kind of curve indicates apparently that you have rolling friction in the material; that the material doesn't slide within itself; the particles roll, thereby bringing new surfaces into exposure constantly.

Our results show that the "coefficient of friction" is a rather inaccurate word under many conditions; it includes structure effects; much of the apparent change in friction due to change of loading seems really due to such factors. We are working on the problem of evaluating these.

A point we wish to bring out is that the material is changing its characteristics throughout the test. We went up to 12 and 13 per cent deformation in these aggregates and found that it was still expanding, the voids increasing. There is a large amount of difference in the extent to which the voids increase in the different materials, but there is increase in practically all of them during the deformation. There is not a constant condition within anything like the deformations usually used in normal tests.

What I wish to point out as a caution is that until it is determined what the effect of the change of pressure is, and establish just what the proper working pressure is, we should be very cautious about interpreting either of these tests, either compressive tests or tri-axial tests. I would say that the method by which we use the tri-axial includes a compressive test; that is, we make a direct relationship between the normal compressive test and the Mohr circle diagram test.

The foregoing remarks comprise the substance of the supplementary material mentioned by Mr. Housel, and perhaps represent all that is advisable to set forth at this time. The subject is so complex that we wish to cover more ground before giving quantitative comparisons of different materials.

Another point appears in some of the papers in that connection. Mr. Hveem uses normally 140 degrees F. for running his stabilometer test, on the assumption that that represents the condition where most of the shoving takes place. This is a rather high temperature. Mr. Vokac, in his paper, endeavors to relate shoving to three different things: To penetration at 77 degrees, which is a fairly normal temperature; to melting point of the material; and to high and low temperatures. The resistance to flow of an asphalt is not in proportion to the penetration at all. The actual relationship is a curve only roughly established. Mr. Vokac gives Class A which was brittle and cracked; penetration 18; Class B, which was satisfactory, neither cracked nor shoved, penetration 29; Class C, penetration 39, which was unstable.

Of course we all know that plenty of roads have been built with asphalts of a much higher penetration in the mixture than 39, which are perfectly stable. It is to be assumed, therefore, there is some other cause for instability than high penetration of asphalt in this case. In actual asphalts that is resistance to flow. Considered indirectly, measured by penetration, Class A has 19.5 megapoises viscosity; Class B has 7.6; and Class C has 7.4. In other words, the asphalt in the cracked roads is just about four times as hard as the others, although the penetration ratio is 1:2.

The melting point in most of these ranges is around 140, 150, 160, and so forth, and when one calculates viscosity under that condition the Ring and Ball melting point in most normal road asphalts is equivalent to a consistency of about 800 penetration. In megapoises that is about 0.01.

It is fairly obvious that, taking it from the point of view of the consistency of asphalt alone, there should be no stability at all worth mentioning in the material at such a temperature. Stability then becomes a function of the aggregate, itself, rather than of the asphalt.

In checking Mr. Hveem's method of testing with our own, we used quite a different principle, although we used the same instrument. We found his stability ratio had a close connection with the coefficient of friction of the aggregate. Therefore, it seems that if we are going to consider these high temperatures and the stability of the pavement under them, the nature of the aggregate has to be very closely studied, because the viscosity of the asphalt becomes so small at such temperatures.

We can throw light on one point. With an asphalt of high viscosity, the coefficient of friction in the mixture is quite low but the cohesion is high. With a soft asphalt the coefficient of friction goes up, while the cohesion goes down. In other words, as the asphalt becomes softer, whether due to temperature or whether due to the use of softer asphalt, a rise in coefficient of friction in the aggregate tends to replace the stability lost by the asphalt. The stability afforded by the asphalt, of course, does not disappear. It still remains, as Mr. Hveem has found, even at those very low viscosities. In other words, you wouldn't get any normal working temperatures where you could throw the asphalt out, and say you still had as stable a pavement.

The general tendency of researches on the consistency of asphalt in regard to stability seems to indicate that, taking it all the year round, the normal temperature consistency is more important. That is, perhaps the periods are too short during which the high temperatures are obtained, or some such factor is present, so that normal temperatures give a better criterion. That, of course, is largely a guess at the present time, based on general indications. This is one of the fields that has to be covered. In other words, what testing temperature should we use to give the best index as to whether a pavement is going to be unstable or not?

If I am not trying your patience too much, I would like to bring up the point of ductility which is mentioned in some of these papers. It has been interpreted that a low ductility is apt to mean a brittle pavement, but the tabulations given in these papers have not very well borne that out. That is, a number of rather systematic penetration drops are shown and are correlated with brittle pavements, but the ductilities don't run consistently in the same direction. I would say some of our own experiments with a beam test show up brittleness very well indeed. On an automatic deflection curve one finds very characteristic transitions from a flexible to a brittle condition. The changes correlate with the consistency but they have no relationship at all to ductility. One can see some point in that from the fact that ductility is measured usually in several centimeters at least, while the amount of extension of an asphalt film which is required to cause a crack is an extremely small fraction of an inch.

So our opinion is that ductility, as far as present knowledge goes, simply doesn't indicate anything in particular except as one of the characteristics which serves to identify an asphalt, but there is a very definite relationship between viscosity and the effect of the asphalt on brittleness in one direction, and lack of stability carried to the extreme in the other direction.

Mr. Housel: Before we leave this subject of mechanical stability, I would like to ask Mr. Endersby one question. Have you conducted any direct shear tests on the same aggregates at the same degrees of compaction to attempt to correlate them with the stabilometer tests?

V. A. Endersby: California. We have not, ourselves, attempted to conduct any shear tests. We have made some studies, including some of Professor Housel's, also. Some curves showing the results, are included in the paper itself. We find that the correlation is fairly close. That is, the shear tests show a similar trend to those in the tri-axial test. Unfortunately, the most comprehensive set of tests we found were done on wet aggregates. While the trends were the same, our own curves were much sharper than with the wet aggregates; this is, the effect of change of pressures seems to be much more marked if the aggregates are dry. Asphalt has a similar effect. The curvature is still there but is much reduced when you introduce the asphalt. However, as said before, other factors than change in friction are present.

Mr. Housel: You say the trends are the same. Specifically, though, do you get the same value of θ, the angle of internal friction, from the shear test that you get from the stabilometer? The trends may be the same, but the values may be different. Do you get comparable values from a direct shear test?

V. A. Endersby: California

We have just one case in that connection that would be closely related, and that is the Ottawa sand. It is very difficult to reproduce a material which somebody else has experimented with. Ottawa sand comes as near to that as anything we can get. We find, in comparing our results with the tri-axial method in this respect with some which Professor Housel performed, also with the tri-axial method, there seems to be a very close relationship. In fact, it seems to be nearly identical when you allow for some differences in void content, and so on. Dr. Housel published a paper some time ago which compares the results of his tri-axial test with shear box tests; this might be interesting to study in that same connection.

Professor Housel's conception of the resistance of these materials is somewhat different. He somewhat questions the conception of internal friction. So do we. We consider it a more complicated phenomenon than most people consider it to be, and probably there is a difference in the initial stability of a material before you start to deform it and the condition which exists afterward; that is, it is moving in the one case and is static in the other.

Mr. Housel: I was trying, Mr. Endersby, to get a definite statement as to whether θ from a shear test compared with θ from a stabilometer test, but possibly it is something we can get together on and not take any more time here.

We have several more papers that should be mentioned. There has been some discussion of Mr. Vokac's paper. Inasmuch as the results of that work have been so widely publicized and you are all familiar with them, I think I will pass over that without asking for any more comments.

Mr. Campbell's "Supplying Vitamins and Eliminating Bugs in Asphaltic Road Materials" was a general paper stating objectives that would be desirable to attain. Mr. Clark, Chief Chemist of the New Mexico State Highway Department, has prepared some supplementary information on that subject, and I am going to ask him to briefly present his results.

R. G. Clark: New Mexico — Probably the best way to summarize this would be to take about five minutes, if I may be permitted to do so, and read part of it. I will eliminate the first part of this paper where I discuss the past uses of cracked and uncracked oil, except to say we have had some pretty sad experience in the State of New Mexico in the use of cracked oil.

Mr. Housel: Thank you, Mr. Clark. Are there any questions or comments from the floor?

Gentlemen, we didn't get started on time, and we are not closing on time. There are two more papers we haven't discussed: One "The Present Status of Construction of Low Cost Bituminous Surfaces," which was more or less a summary and review of current construction practice, which I don't believe we need spend any time here in discussion; and, "Recent Developments in Soil Stabilization" by Dr. Winterkorn, which we can discuss or not as we see fit.

I might remind you that we are going to have a round table discussion Thursday morning, following the practice that was initiated last year at Biloxi, where all the loose ends from the various sections and unanswered questions and discussions were taken up. That proved to be such a valuable meeting that it has been incorporated in this year's program. So I am not particularly disturbed in having such a short period for discussion now as there is the possibility of leaving any of our discussion on soil stabilization over for our round table discussion. However, if there are any of the members on the floor who would like to ask Dr. Winterkorn any questions at this time, or make any comments, we would be very glad to have you do so.

V. G. Gould: Iowa — It will take only a minute to say what I have in mind. I want to mention to Dr. Winterkorn that I take exception to his statement, which I believe I understood correctly, that it has been determined that P.I.'s in excess of six are unsatisfactory. I will discuss the matter in more detail at the round table.

Mr. Housel: I think it would be a good idea to take that up at the round table, Mr. Gould, because in the first place he was quoting some conclusions given from the study made by the Bureau, and as my time was also getting short I didn't give all the conclusions. I think if all those conclusions were given it might very materially alter the implication of that particular one. That will be a very good subject for you to keep in mind for the round table discussion, and we can go into that matter in much more detail. Soil stabilization is such a large subject that I really hesitate to open it up at this late hour. It is now past our closing time, so let us keep soil stabilization in mind for the round table discussion. I believe that relieves you of further discussion at this time, Dr. Winterkorn

I take it that adjournment is in order, gentlemen.

Papers Prepared

on

Research as Applied to Construction and Maintenance of Bituminous Surfaces

for

The Montana National Bituminous Conference

Hardening of Hot-Mix Asphaltic Materials

By

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"Hot-Mix Asphaltic Materials" as used in this paper refers to asphalt cements that are added to graded aggregates in making "hot-mixed" bituminous mixtures.

The customary procedure, in making these "hot-mixed" bituminous mixtures, is to heat the asphalt cement and add it to the heated aggregates in predetermined proportions, mixing thoroughly for a specified time, in what is known as "pug-mill" or "rotary" type mixers. Sheet Asphalt and Asphaltic Concrete are examples of these types of mixtures.

Considerable interest has been evidenced in the subject of hardening of Hot-Mix Asphaltic Materials, and rapid strides are being made in the development of methods to control the amount of hardening, or at least control the hardening to such an extent that Hot-Mix Type Pavements may receive more consideration and be adopted for highway use to a greater extent than in the last few years.

Permanence cannot be expected in Asphalt Pavements, or any other type of pavement, regardless of whether they are laid by the Cold-Mix or the Hot-Mix Method, because they are all subject to deterioration, through the agencies of air, water, expansion and contraction, the same as any other structures. Maintenance is therefore necessary on all structures if the investment is to be protected.

It has been shown by various investigators that the asphaltic cement in a pavement will gradually harden (as measured by the penetration test). This hardening of the asphaltic cement is recognized as one of the reasons for cracks eventually developing in a Hot-Mix Pavement.

Hubbard and Gollomb (1) report that 30 penetration of the asphalt cement is about the critical limit for the Eastern states, whereas Powers (2) of Arizona gives cases in which there was no cracking when the recovered asphalt cement had as low as 15 penetration.

The fact remains, however, that there is a gradual hardening of the asphaltic cement in Asphalt Pavements, and when it does reach a certain degree of hardness, cracking and other forms of deterioration will set in.

Bateman and Lehmann (3) showed that 20 pound batches of Sheet Asphalt mixture, when mixed for 9, 8, 6½, 5 and 2½ minutes (using 46 penetration asphalt) at 250, 300, 350, 450 and 500° F., respectively, gave recovered asphalt having penetrations of 40, 36, 35, 33 and 24, in the order named.

Rader (4) records cases where the extracted asphalt in a Sheet Asphalt pavement (about 8 years old) gave a penetration of 34. In the original hot-mix a 56 penetration asphalt was used. He also cites a second case of a pavement (about 11 years old) where the extracted asphalt had a penetration of 13, as compared with a penetration of 42 in the original hot-mix.

Raschig and Doyle (5) recorded a sufficient number of instances to show that a considerably lower penetration is obtained on the extracted asphalt from various pavements, than the penetration of the original asphalt used in the preparation of the hot mixture.

Covert (6) reports recovered asphalt of 24 penetration in a pavement (5 years old) when 43-45 penetration asphalt was initially used. He finds most of the penetration loss occurred in the mixer box. Using an 80 penetration asphalt with a mixing time of 45 and 15 seconds, a loss in penetration of 12-15% was obtained.

Miller, Hayden and Vokac (7) examined and studied a large number of pavements, classifying them as Group ''A'' (where cracks developed); Group ''B'' (sound type which had neither cracked nor shoved), and Group ''C'' (where the mixtures were rich enough in bitumen to shove); and found that the hardening, expressed as the penetration loss on the recovered bitumen, was 63, 43 and 29%, respectively, with Oil Asphalt Cements. The average age of the Groups A, B and C Pavements were, respectively, 10, 9 and 14 years.

Most of this hardening of the asphalt cement occurs during the mixing of the asphalt cement and hot aggregates and the time interval before placing on the road, and this was undoubtedly true in the above cases. Some of this hardening can be accounted for when keeping the asphalt cements at unusually high temperatures before actual use. Careful attention should be given to the heating of the Asphalt Cement, because all types of asphalt differ in fluidity at the same temperature.

Most Hot-Mix Specifications allow for the heating of the aggregates to as high a temperature as 375° F., the Asphalt Cement to 250 to 350° F., and also carry a minimum temperature of 275° F. for the mixture before it is placed on the road. The high void content of this uncompacted mixture, together with its temperature, allows the air to act on the films of asphalt, thus hardening it and changing the characteristics of the asphalt cement. Careful attention to the temperature of mixing, time of mixing and temperature at the placing of the aggregate-asphalt mixtures, will result in a large decrease in the hardening of the asphalt.

The action of air on asphalt is known as ''oxidation.'' Air in contact with asphalt at different temperatures effects a change in the asphalt by increasing its softening point, lowering its penetration (or making it harder) increasing the amount of asphaltenes; in other words, it changes the characteristics entirely. To make this clear, the following results, when blowing air through a soft residual asphalt of 100° F. (Ball and Ring) softening point, were obtained:

Softening Point, ° F. (B. & R.)	101	113	118	140	178
Penetration at 77° F. (100 g., 5 sec.)	179	112	85	46	24
% Insoluble in 86° Naphtha (% asphaltenes)	12.3	18	19.1	23.5	29.2

The higher the temperature, the faster this oxidation proceeds and, necessarily, the lower the temperature and the thicker the film, the slower the action is.

Asphalt Cement, even in the absence of air, develops another form of hardening, due probably to certain cohesive forces within the asphalt. This is sometimes referred to as ''age hardening.'' However, heat and kneading action disturb these cohesive forces. This kind of hardening can be illustrated by the usual penetration test, allowing the sample of asphalt to remain undisturbed for various periods before determining the penetration at 77° F. without reheating the samples.

Traxler and Schweyer (8) measured this change or ''age hardening'' by determining the viscosities at different intervals of time and developing an aging index. They found, also, that reheating of the asphalt, after ''time hardening'' for several months, caused it to return to within a few per cents of its original viscosity.

Recognizing the fact that most of the hardening of the asphalt film has been, and is, due to oxidation, more careful attention is being given to control and use of lower mixing temperatures and time intervals, as well as avoiding the placing of the hot-mixes on the road at unusually high temperatures. This will prevent about 50% of the hardening of the asphalt that takes place during construction of the road. Hubbard and Gollomb (1) show, by laboratory tests on Sheet Asphalt Mixtures, that time and high temperatures accelerate the hardening of the asphalt.

Very little information is available on the progressive hardening of the asphalt while the pavement is in actual service. This information will undoubtedly be available from year to year, as Michigan, Ohio and other States and investigators release their results on the detailed and thorough work on this subject. It is a recognized fact that the thinner the film of asphalt, the smaller the percentage of voids should be in order to prevent oxidation by air as much as possible.

With the Abson (9) method available for the recovery of asphalts from the extracted bitumen in a pavement, the progressive hardening of asphalt cements can be followed more readily, and with its use, closer control can be exercised on Hot-Mix Type pavements.

Baskin and Lebel (10) report a series of tests using 60/70 penetration asphalts of varying degrees of susceptibility. They prepared laboratory molds of Sheet Asphalt and allowed them to age for a period of 5 months. They report that the lower susceptibility asphalts showed a greater resistance to hardening. Inspection of their data indicate a recovered bitumen of 48 penetration on the compressed molds (using a low susceptibility asphalt) mixed at 350° F. and allowed to age 5 months. The original 62 penetration asphalt had a ductility of only 34 centimeters at 77° F. Using the same mixing temperature and procedure, a 66 penetration asphalt (of higher susceptibility) of 110+ ductility at 77° F. hardened to 25 penetration after a 5 month aging period.

Hubbard and Gollomb (1) prepared Sheet Asphalt mixtures at a temperature of 325° F. using asphalts representing nine sources. After exposing these mixtures, in loose layers, at 325° F. for ½ hour, the oil asphalts of approximately 50 penetration showed a hardening in penetration of 19 to 36%.

Skidmore and Abson (11) in reporting work on "Progressive Hardening of Asphalt Cements in Paving Mixtures," state that, irrespective of source, 10 to 20 points in penetration of normal hardening is indicated by laboratory results under good plant practice.

In order that the finished pavement will have an asphalt incorporated into it, as close to the penetration originally desired, softer asphalt cements have been used, thus allowing for the hardening during heating, mixing and placing. The use of softer asphalt cements in Asphaltic Concrete on large scale construction has been described by Delano (12) on work done by the Massachusetts Highway Department. Withycombe (13) has outlined California's experience and work, also using softer asphalt cements.

The attention now being given to Asphalt Cements in Hot-Mix Construction, together with the knowledge of their characteristics and methods of handling, as being developed by the various Highway departments, undoubtedly will result in determining the proper consistency of Asphalt Cements to be used in various localities, thus compensating not only for the loss in penetration of the Asphalt Cement during actual construction, but for the gradual hardening during the life of the pavement.

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Quantitative Methods of Measuring Adhesion of Bituminous Materials

By

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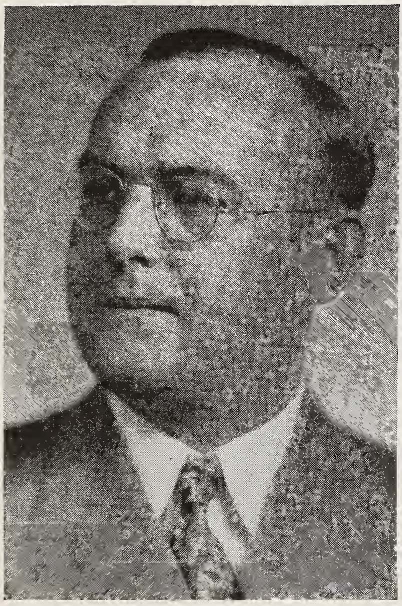

A quantitative method presupposes a qualitative knowledge of the phenomenon under consideration; it measures the extent or intensity of this phenomenon. The phenomenon in which we are interested here is the stripping of bituminous coatings from mineral aggregate under the action of water. There are two major variants of stripping, one leaving the stripped surface of the mineral washed clean, the other leaving the mineral after loss of almost all the bituminous material with a very thin coating of apparently deadened oil. In the first case the stripping appears to be caused entirely by the action of water, for the second case a preparing effect of the adsorbed oxygen on the bitumen film has been suggested.

At the last two meetings of this conference attempts were made to cover as completely as seemed feasible the theories which demonstrate why materials adhere to each other and how and why the relative strength of this adherence varies for different materials (1, 2). Every theory deals to quite an extent with abstracts. In the formation of abstracts certain natural phenomena are more or less emphasized, others are pushed in the background as of minor importance. While certain of these secondary phenomena may not be of sufficient weight to change the general structure of the picture, or may be implied in its general structure and visible for the theoretical chemist but not for the highway engineer, they may be of the utmost importance in the field of highway construction. To develop quantitative methods of measuring adhesion of bituminous materials or to render quantitative the methods now available, details must be drawn of certain parts of this general picture of adhesion phenomena which was sketched in the preceding reports.

In the general theory the mineral particles are usually treated as having clean surfaces with an energy or electric pattern according to their composition and crystal structure. It is quite often overlooked that mineral aggregate even if free from dust and other visible impurities contains films of material to which it has been exposed in its past and present existence such as moisture, air, etc. The moisture films especially are rather tenaciously held and may be present even at the highest drying temperatures used in bituminous practice. The presence of these strongly held moisture films in itself does not necessarily represent a calamity since they may be held with such great energy by the mineral that they become part of the latter and in a way it does not matter whether our bitumen sticks directly to the mineral or indirectly by means of an interposed moisture film. This is entirely true as long as the moisture film does not change its properties, as long as it really sticks to the mineral in a solid or semisolid condition and does not acquire the properties of more or less free but entrapped water. In the last sessions of the Association of Asphalt Paving Technologists (3) the problem has been discussed what entrapped water may do to a pavement as a whole. Water entrapped around a mineral particle which is coated by a bituminous film may have the same effect, revealing itself ultimately as stripping.

The question, therefore, arises whether or not adsorbed water may change into trapped water; and if so, under what conditions and to what extent is this liable to take place. Since the thickness of adsorbed water films is in general very small, it is difficult if not impossible to study this question with the help of larger sized aggregate. To obtain measurable values for the amount of adsorbed water, a material with a very large surface to mass ratio has to be chosen. Materials of this kind are the well known soil colloids. The amount of this increase of specific surface (surface/mass) can be visualized from the data in Table I, showing the increase in number of particles and in surface by subdivision of a cube of 1 mm side length.

TABLE 1

Increase in number of particles and amount of surface by subdivision of a cube 1 mm side length

Length of side of cube in mm	Number of cubes	Increase in number	Total surface in mm^2	Increase in surface factor
1,000	1	1	6	10^0
0,100	1,000	1x10^3	60	10^1
0,010	1,000,000	1x10^6	600	10^2
0,001	10^9	1x10^9	6000	10^3

The behavior of adsorbed water under varying conditions and, therefore, its possible change into free (entrapped in the case of bituminous coating) water has already been studied by soil scientists and we can make good use of their findings.

Baver and Horner (4) investigated the influence of chemical composition of soil colloids on the amount of water which they can adsorb and hold at different temperatures and under different conditions. In other words, they studied how much adsorbed water is set free by the colloids under different conditions. Table 2 shows some results of this work.

TABLE 2

Water loss at different temperature intervals by colloids which had been permitted to come to equilibrium with moisture over a 30% aqueous solution of sulfuric acid

	Temperature Intervals							
Colloid	x-60	60-110	110–M.P.	60-M.P.	M.P.	Sio2 R203	Tot. ads.	% of water content removed by drying at 60° C.
H.Benbonite	19.40	2.14	3.45	5.59	400	5.00	24.99	77.5
Lutkin E.	20.00	1.91	1.78	3.69	225	4.82	23.69	84.5
Putman E.	18.41	1.97	2.14	4.11	275	4.05	22.52	81.7
Susquehanna E.....	15.06	1.56	2.39	3.95	150	2.88	19.01	79.5
Cecil E.	6.43	0.67	.29	.96	125	1.82	7.39	86.5

Water loss in % water per dry weight of soil

x designates adsorption over 30% sulfuric acid.

These data show that about three-fourths of the water which is adsorbed by the colloids over a 30% H_2SO_4 solution (relative humidity 75%) is set free at 60° C. If this free water is entrapped by a bituminous film, it might have some fun. The data referred to while being of great significance have the drawback that the water-vapor pressures existing at the higher temperatures under the experimental conditions are not definitely known. Baver and Winterkorn (5) in further pursuing this problem determined the water held by soil colloids at constant water vapor pressures, but at different temperatures. Their results are given in Table 3.

TABLE 3

Water adsorption by different aluminosilicates as a function of temperature. (Vapor pressure 30 mm Hg, relative humidity 94% at 30° C.)

	Temperature in centigrades			
Colloid	30°	40°	50°	80°
H-Bentonite	32.7	13.0	7.51	2.12
H-Putnam	25.8	10.8	5.7	2.3
H-Davidson	21.4	3.5	2.6	1.2
H-Permutite	28.4	16.9	4.9	3.1

% by weight H_2O adsorbed

These data corroborate the results obtained by Baver and Horner, and serve to emphasize still more the fact that at low temperature relatively large amounts of moisture may be held by dry appearing aggregate. A large part of this moisture may be set free at elevated temperatures which are attained by the pavement in service. This liberated water is liable to stir up trouble. The implications of these phenomena for the stripping problem are evident. They are that mixing temperatures above 60° or 80° C. are preferable to lower ones, and that heating and cooling of an aggregate before mixing will not do so very much good if the aggregate is not prevented from absorbing moisture anew, corresponding to the lowest cooling temperature reached. This implies moisture control of the atmosphere in the cooling chamber. The implications for quantitative stripping determinations are:

a. To obtain reproducible data, the always present moisture film on the aggregate must be permitted to come to an equilibrium with a definite moisture pressure at a definite temperature, before a bituminous coating is applied.

To obtain a quantitative measurement of the stripping tendencies of an aggregate in a road, the temperature and vapor pressure to which the aggregate is exposed before coating has to comply as closely as possible to the conditions obtaining on the job.

If these stipulations are observed, any of the wash tests or water shaking tests which have been described in the literature and have been enumerated in the proceedings of the last two years can be used for quantitative work. The data given above also bear on bituminous practice insofar as they indicate that reduction of the water content of the mixture before consolidation to that amount which the mineral particles still hold adsorbed at the highest possible temperature which the pavement will reach during service, will go a far way in reducing stripping trouble caused by water. These considerations also give an idea why stripping trouble was almost unknown before the advent of cold construction (6). But, what about moisture coming in after the pavement is completed? Since one of the purposes of the use of bitumen is to keep water out of the pavement, this can happen only in faulty constructed pavements. The affinity for water of the common minerals or of the thin water film adsorbed by the minerals is not so much different in energy from that for organic materials of the bituminous type. Data on the heat of wetting generated by benzol and water respectively on air dried soil colloids have demonstrated this (7). Therefore, the longing for water by a well coated mineral particle is, in general, not sufficient to pay for the energy required for water to diffuse through a substantial bituminous film. For all practical purposes water can reach the surface of the mineral only if the bituminous film is extremely thin or if the latter has been hardened and cracked and permits the entrance through the created fissures. In hot constructed pavements this can be avoided by good paving practice as is being recommended by all experts. It is in the case of cold constructed pavements especially those with very thin films as obtain in bituminous soil stabilization that affinity troubles make themselves felt most at the present time. With thin films, which are at least partly permeable to free water, and the latter being often available in stabilized soils, opportunity for water adsorption and loss of stability is often given in such systems. It is in this field also that the chemical differences of the minerals which show themselves not so much in the energy of water adsorption but rather in the thickness of the water film which is adsorbed at low temperatures, make themselves most conspicuous. However, discussion of these phenomena belongs in the field of soil stabilization. For materials which have been mixed at elevated temperatures and have been well designed and well laid chemical differences in the aggregate play only a secondary role (6). The influence of the chemical character of the minerals on the stripping resistance of cold constructed pavements indicates the beneficial use of chemical treatment such as has been proposed in the literature, or the selection of aggregate which does not show appreciable differences between the thickness of the water films which it adsorbs at low and high temperatures respectively. Accordingly, this chemical character is of obvious importance if wet aggregate is encountered during construction, or if asphalt emulsions are employed. In these cases the case of covering the wet aggregate by the bitumen is a general indicator of the desirable or undesirable properties of the water film which is held by the aggregate. The general objections against using this phenomenon for testing purposes have been enumerated previously on the occasion of the discussion of the preferential wetting test (2). These objections become somewhat milder if only enough water is used to wet the aggregate. In this way the coating test was applied by McLeod (8) to determine the effectiveness of certain treatments (formation of metallic soaps on surface of hydrophilic aggregate) on the adhesiveness of bitumen. The ease and speed with which the mixing test can be performed, and the interesting results which had been obtained by McLeod caused Winterkorn and Eckert (9) to compare results of the mixing test with those of the wash test obtained on identical samples. The experimental investigation referred to showed that:

1. The ease of covering moist sand with a bituminous material by simple mixing might be a general indication, but is not an exact and dependable measure of the adhesion relationship between the bitumen and the sand;
2. The best agreement of the mixing test data with those from the wash test occurs in cases where wetting agents of soap-character are used in conjunction with Al, Fe and Pb-ions, i. e. with those ions which are generally recognized as favorably affecting adhesion of bitumen to aggregate. For such cases, and where aggregate must be coated in wet condition with the help of soap type activators the McLeod test appears to be applicable;
3. The mixing test cannot be substituted for the wash test if definite information on the adhesion relationship between bitumen and aggregate is desired.

In order to obtain a reliable and quantitative measure of the stripping tendencies of a wet coated aggregate, the coated material which is to be tested should be allowed to come to the same moisture content which it possesses on the field at the time of compaction. This procedure automatically implies a certain factor of safety since the adhesivness of the bitumen to the stone is liable to increase with time.

In this connection it cannot be emphasized often enough that the wash test measures only the resistance to stripping of the components of the pavement. The resistance of the pavement system as such is not a simple summarization of the properties of its components but is, to a very large extent, determined by design. Methods applicable to representative pavement samples are the swell test, the limitations of which have been described previously and stability tests on samples before and after saturation with water. At the present time no general agreement has been reached as to the method by which this saturation should be allowed to take place. Saturation by immersion is somewhat indefinite because of the influence of swelling and occluded air in the sample. Saturation by capillary ascent is not always permitted to come to conclusion in some of the present testing methods. Reduction of the atmospheric pressure in both the immersion and capillary tests appears to be of promise in obtaining speedier and more reliable absorption conditions. Storing of the samples in a moist closet such as used for Portland cement specimens might also provide a satisfactory method for the adsorption part of the stability test. A coordinated study of the effectiveness of the different methods of exposure to water by an agency such as the A.S.T.M. or other appears to be highly desirable. Stability tests on moisture saturated specimens should be run, not only at room temperature but also at the highest temperature which the pavement is liable to attain in service.

Mention has been made in the beginning of this paper of the effect on adhesion of the air film adsorbed by a dry mineral particle. Since the adsorption capacity of a mineral for air varies with the temperature, and since change of adsorbed to free air, especially at elevated temperatures, involves quite a large volume change, adsorbed air will act in very much the same way as adsorbed water. Besides, adsorbed air, especially its oxygen component, might oxydize some of the bitumen (with or without catalytical help by the mineral surface) with bad effect on the consistency of the bitumen. The combined chemical and physical effect of adsorbed air may prepare a way for moisture to enter under unfavorable conditions and to complete the work of destruction. It is logical that this effect would be most noticeable in our more arid regions and it is certainly no accident that New Mexico has paid close attention to this problem. While I am not at liberty to reveal information which Mr. Powers has been kind enough to make available to me, I might say that very interesting work is, at present, being done on this problem by Mr. Powers. Certainly, we shall soon learn more about the bearing of air adsorption on the stripping problem.

Summarizing, it can be stated that the best available methods for testing the resistance to stripping of a bitumen coated mineral are the different modifications of the wash test in which the coated mineral particles are shaken in the presence of water. These methods furnish quantitative data if the conditions to which the aggregate is subjected before, during and after the coating before testing are standardized as well as the actual test procedure. This standardization must be of a kind to insure definite equilibria between the surface of the mineral and the gaseous and (or) liquid phases with which it comes into contact before being submitted to the action of water during actual testing.

To be of practical value the standardized procedure should represent as closely as possible the conditions obtained on the road.

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Recent Developments in Soil Stabilization

By

HANS F. WINTERKORN

Soil stabilization is coming of age. After a busy childhood and a typical adolescence with overconfidence and inferiority feelings chasing each other, she is now going to finishing school. The awkwardness of youth is being stripped off, she has to let go of some of her dearest aspirations but her inherent qualities are being carefully developed; the mixed loudness and shyness is being turned to a modest but assured show of her perfections. While her birth was a result of the needs and dreams of the practical engineer and while her youth was a rough one spent mostly in the fields her finishing school is largely under the supervision of a worldwise industry which knows how a lady ought to behave in public.

At the present state of development, the need for a classification of the different types of soil stabilization becomes more and more obvious. For some time the Missouri State Highway Department has been differentiating between—

1. Mechanical Stabilization, including simple compaction and preparation of compacted systems, graded soil and aggregate materials.
2. Chemical Stabilization, embracing all such systems the components of which cannot be completely separated by physical means into their original condition after the stabilized system has been finished: stabilization by means of bituminous materials, Portland cement, etc.
3. Mixed stabilization, such as clay-gravel stabilization with additional bituminous treatment.

Roediger and Klinger (1) have recently proposed the following classification:

1. Compaction stabilization—which entails simple densification without admixture.
2. Waterproofing stabilization—which includes the incorporation of water-insoluble binders with soil or soil aggregate.
3. Dilution stabilization—in which the amount of active clay or silt is reduced by the addition of suitable soils or aggregates taking into account the attainment of a specified grading.
4. Cementation stabilization—in which a hydrating material capable of developing setting properties is incorporated with soil or soil aggregate.
5. Chemical stabilization—which includes the treatment with water-soluble salts of soil or soil aggregate with the result that the

moisture content of the mixture is maintained at a definite value, or so that a chemical reaction may take place.

It is realized that any classification has difficulties coping with borderline cases. (Of these soil stabilization is especially rich). It is also realized that certain industries might feel the need or the desirability of coining special names for the achievements of their products or to put them into a class for themselves. Considering all the points in question it appears to be most adequate to establish a frame classification on a scientific basis and to make subdivisions which recognize the desires and needs of the industry.

Examination of the classification by Roediger and Klinger reveals that the subdivisions are not of the same intrinsic weight. Especially the use of the term ''chemical stabilization'' restricted to the addition to soil of water soluble salts is open to great criticism. Research work in stabilization has shown the great importance of the chemical characteristics of the materials involved in bituminous stabilization, not to speak of the obvious chemical nature of the cementation stabilization.

Using the generally accepted meanings of scientific terms and respecting the legitimate desires of industry the following classification might be proposed:

1. Mechanical stabilization: Effect mainly due to manipulation and compaction of the soil. This includes possible use of water and salt solutions which are not entirely alien to the soil and which while able to change the physical properties of soils do not render them fundamentally different systems.
2. Physical stabilization: Preparation of graded soil mixtures by adding coarse or fine soil constituents which do not lose their physical identity in the system and which can be reclaimed by physical means such as screening, flotation, sedimentation, etc. Dilution stabilization, clay-gravel, sand clay systems and such, with or without addition of salt solutions as under 1.
3. Chemical stabilization: Addition of materials which because of their own chemical character and that of the soil constituents react and form definite systems from which the original components cannot entirely be reclaimed by physical and mechanical means. Cementation stabilization hydraulic and other inorganic cements.
4. Physico-chemical stabilization: Where the effectiveness of the treatment is due to both physical and surface-chemical factors which can not be separated in a clean way, while the importance of each of these factors may vary. An example for this is bituminous stabilization, in which both the chemical character of the bitumen and the soil and also the physical character of the bitumen play important roles. The bituminous stabilization, of sands borders more on physical stabilization, while that of fine soil materials is governed to a great extent by chemical factors.

Of course, mechanical, physical, physico-chemical and chemical factors play a role in all types of soil stabilization. If under these conditions one wants to give a name at all, it should be given in accordance with the most important factor or influence. It might be good salesmanship, but, certainly, it is not scientific to describe a physical soil stabilization involving the addition of sand and gravel to a soil, and conclude with the words "the soil is then stabilized by addition of so and so much of a soluble chemical." The proposed system shall be used in the following.

Mechanical Stabilization:

The determination of the optimum moisture content for soil compaction as developed by Proctor has been one of the most fruitful contributions to the use of soils for engineering purposes. The principles of compaction are employed not only in the construction of dams and fills or in the general preparation of soils as foundation material, but they are governing also all the other types of stabilization. Proposed and accepted specifications for clay-sand, clay-sand-gravel, bituminous and cement stabilization include Proctor compaction as an essential feature.

Because every highway engineer is or should be intimately acquainted with the principles and the practical scope of soil compaction a detailed treatment did not appear warranted at this place. However its importance can be judged by the fact, that, like a red line, the theme of compaction runs through the rest of this paper.

Physical Stabilization:

Last year there was an interesting discussion at this meeting concerning the desirable properties of the soil binder in physical stabilization. Especially the problem of the right Plastic Index was very much in the foreground. In accordance with the then available knowledge the discussions were more or less on a qualitative basis. In the meantime two excellent pieces of research have been reported on this problem by the Bureau of Public Roads. These are:

1. A study of sand-clay materials for base course construction.
2. A study of sand-clay-gravel materials for base course construction by C. A. Carpenter and E. A. Willis (2).

Since these investigations represent the greatest progress that has been made for quite a while in the study and rationalization of design in this type of stabilization they will be treated here in relative detail.

In the first paper two series of sand-clay mixes were investigated, one representing essentially a constant gradation but a variation of the Plastic Index from 0 to 18, the other maintaining approximately the same Plastic Index but with variation of gradation. The service behavior of the mixture was studied by means of the circular track. Supplementary tests were also performed ultilizing—

a. Compaction studies according to Proctor;
b. Compaction by means of a vibrating machine described by Pauls and Goode (3);
c. Determination of "stability" by means of the Hubbard-Fields apparatus:
    1. On cores taken from the track;
    2. On molded specimen covering a range of moisture contents.
    3. On molded specimens with a variation of compression loads at constant moisture content.

TABLE 1

GRADING, SOIL CONSTANTS AND PROPERTIES OF SAND-CLAY BASE COURSE MATERIALS

	SERIES 1					SERIES 2					
SECTIONS:	1	2	3	4	5	1	2	3	4	5	6
GRADING:	%	%	%	%	%	%	%	%	%	%	%
Passing No. 10 Sieve	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Passing No. 20 Sieve	91.4	90.5	91.7	90.0	90.6	90.2	87.7	89.1	93.8	82.9	67.3
Passing No. 40 Sieve	70.6	65.9	67.7	67.1	67.3	69.2	64.3	64.5	80.2	50.6	47.5
Passing No. 100 Sieve	35.8	31.0	31.6	33.4	33.7	51.2	37.7	26.0	53.0	25.0	33.8
Passing No. 200 Sieve	25.1	25.5	26.9	28.6	29.1	44.7	31.0	20.1	40.8	20.2	29.3
Passing 0.005 mm.	10.5	14.5	18.0	20.5	21.5	16.0	13.0	13.0	16.0	12.0	12.0
Passing 0.001 mm.	6.0	9.5	12.0	14.0	15.0	10.0	8.0	9.0	10.0	8.0	8.0
Dust Ratio	36	39	40	43	43	65	48	31	51	40	62
TESTS ON –40 MATERIAL:											
Liquid limit	18	20	25	28	33	21	19	21	21	20	20
Plasticity Index	0	5	9	13	18	4	4	5	6	6	4
Behaviour:	OPERATION					OPERATION					
Compacting Base	good	unstable	slightly unstable	unstable	good	good	good	unstable	good	unstable	slightly unstable
Comp. Base and Surface Tr.	good	slightly unstable	slightly unstable	slightly unstable	good	good	good	good	good	good	good
Test with distr. traffic	good	slightly unstable	good	slightly unstable	failed	failed	failed	good	failed	good	good
Test with conc. traffic	good	good	good	slightly unstable				good			slightly unstable

TABLE 1
(Continued)
GRADING, SOIL CONSTANTS AND PROPERTIES OF SAND-CLAY BASE COURSE MATERIALS

	SERIES 1					SERIES 2					
SECTIONS:	1	2	3	4	5	1	2	3	4	5	6
do. 160,000 to 176,000 trips	good	good	good	unstable failed							
176,000 to 248,000 trips	good	good	some movement								
248,000 to 266,000 trips	good	good	some cracking								
Density by vibration	80.6	80.8	79.6	78.5	76.3	79.6	81.9	80.5	78.8	81.4	82.3
Final density in track	78.8	80.8	79.5	78.1	76.2	74.7	76.1	79.6	74.9	81.2	80.6
Density Proctor Comp.	74.2	76.2	76.2	75.1	74.5	75.0	76.5	75.3	75.9	76.2	77.8
Change in Hubbard-Field stability per 1% change in water content betw. max. stab. and max. dens.	6 lb.	210 lb.	229 lb.	634 lb	795 lb.	1,800 lb.	847 lb.	181 lb.	1,100 lb.	55 lb.	176 lb.
REMARKS:	excellent throughout compaction and test period	unstable during compaction, excellent throughout test period	fairly stable during compaction, generally satisfactory throughout test period	unstable during compaction failed under test traffic	stable during compact'n failed under test traffic	stable during compaction failed under test traffic	stable during compaction failed under test traffic	unstable during compaction excellent throughout test period	stable during compaction failed under test traffic	unstable during compaction excellent throughout test period	unstable during compaction generally satisfact'ry during test period

The gradation of the test mixes and their behavior on the circular track and in other tests are shown in Table 1. The results of the experiments can be summarized as follows:

1. The circular track can be used to great advantage in testing and classifying clay-sand mixtures.
2. For best service of a clay-sand system, both grading and plasticity must be controlled.
3. The failures which occurred in Series 1, were caused by excessive plasticity.

   The failures occurring in Series 2 are traceable to improper grading.
4. A doubtful behaviour of the clay-sand system is always characterized by a swing of the gradation curve toward finer materials, it is immaterial whether this swing occurs immediately below the No. 10 sieve or below the No. 200 sieve fraction.
5. The plasticity index of the –40 material should be below 6 for satisfactory performance.
6. Poorly graded materials are less stable than dense graded ones.
7. Repeated experience has shown that if the material which is retained on the No. 10 sieve does not exceed 35% the system retains essentially the properties of the mortar (–10) fraction.
8. The following specification is recommended for bases under a bituminous surface treatment:

	Minimum	Maximum
Percentage of material passing No. 10 sieve	65	100
Percentage of material finer than No. 10 sieve passing:		
No. 20 sieve	55	90
No. 40 sieve	35	70
No. 200 sieve	8	25
Percentage of material finer than No. 40 sieve passing:		
No. 200 sieve (dust ratio)		50
Liquid limit (material finer than No. 40 sieve)		25
Plasticity Index (material finer than No. 40 sieve)		6

The following conclusions are offered in the paper referred to:

1. Control of grading, particularly by limiting the amount of material passing the No. 40 and No. 200 sieves is essential to insure satisfactory stability; the control in grading the –200 material is aided by establishing and maintaining a maximum limit for the plasticity index.
2. A plastic index larger than 6 may result in failure under unfavorable moisture conditions.

3. Excessive amounts of dust even if not producing a large plastic index should be avoided, since such material retards compaction and is inducive to failure under unfavorable moisture conditions.

4. Thorough compaction must be obtained if trouble is to be avoided. As a measure of the desirable compaction should serve that obtained by vibration rather than that indicated by the Proctor method. Lack of thorough compaction is much more dangerous for plastic than for harsh mixes with a zero plastic index. The latter may never become particularly dense but may show good servicability.

5. Early difficulties in compacting mixtures possessing acceptable gradings and plastic indices cannot be taken as indicating poor quality in service. Rather the opposite seems to be the case.

6. Compaction of base courses having plastic properties should be completed to essentially maximum practical density before surface course is applied, because movements in the base course which are common during compaction of good materials will cause damage to prematurely constructed surface courses.

7. Using the Hubbard-Field stability test, the rate of stability change per per cent of change in moisture content, over the range of moisture contents where stability is falling while density is increasing, appears to be highly significant as a means of predicting service behaviour.

TABLE 2

	Fraction of total aggregate passing No. 10 sieve	Plasticity index of fraction passing No. 40 sieve	Behavior of section under traffic
SERIES 1	%		
Section 1	53.5	0	satisfactory
Section 2	47.9	5	satisfactory
Section 3	47.9	9	satisfactory
Section 4	50.3	11	essentially satisfactory
Section 5	47.0	16	failed
SERIES 2	%		
Section 1	42.3	0	satisfactory
Section 2	65.0	9	failed
Section 3	31.9	8	satisfactory
Section 4	41.9	7	approached failure
Section 5	38.0	6	satisfactory
Section 6	66.1	7	failed

The work on:

A study of sand-clay gravel material for base course construction followed the same general lines as the one just described. A noteworthy item was the determination of the needed amount of mixing water on the basis of the pore space obtaining in the mixtures after thorough compaction by vibration of the dry material. Other features were the determination of the frost heave in the moist systems by means of dry ice, and the use of a new instrument for the determination of longitudinal profiles. Table 2 shows the quantity and character of mortar fractions of track sections, and behaviour under traffic.

As a result of the study the following specifications were recommended:

TABLE 3

	TYPE B BASE COURSES			
	B-1, 1 inch max. size		B-2, 2 inch max. size	
Percentage passing:	Minimum	Maximum	Minimum	Maximum
2 inch sieve				100
1½ inch sieve			70	100
1 inch sieve		100	55	85
¾ inch sieve	70	100	50	80
⅜ inch sieve	50	80	40	70
No. 4 sieve	35	65	30	60
No. 10 sieve	25	50	20	50
No. 40 sieve	15	30	10	30
No. 200 sieve	5	15	5	15
% –40 material passing No. 200 sieve		50		50
Liquid limit (–40 material)		25		25
Plasticity index (–40 material)		6		6

The following conclusions can be derived from this important investigation:

1. To obtain good clay-sand-gravel bases the gradings and the plasticity indices of the materials must be controlled.
    - (a) The grading of the coarse fraction is of greatest importance if the mortar fraction decreases below 40 per cent.
    - (b) The grading and the plastic index of the mortar fraction is of eminent importance if the mortar fraction exceeds 40 per cent.
    - (c) A plastic index limit of 6 should not be exceeded, higher plastic indices render the systems conducive to failure due to softening and frost heaving. A plastic index of 5 in a well graded mixture is preferable to a zero or intermediate one.
2. Thorough compaction of the mixture is essential for good performance on the road. The bottom layers must be compacted as thoroughly as the top layers. The final compaction should be comparable to that obtained in the vibrator not to that obtained by the Proctor procedure.
3. The water requirements for compaction can be arrived at by adding 1.5% to 2% water to the optimum amount obtained in the Proctor

test for the minus-10 materials, or by determining the voids volume of the mixture after dry compaction by vibration. The compaction operations shall continue during the loss of the excess water. For non-plastic mixes the amount of water employed does not have to be limited, but care must be taken that the subgrade is not softened by excess water.

4. It might be desirable to establish a separate specification for essentially stone or gravel base-course materials, having satisfactory gradings but very low soil-mortar contents, to allow the use of such materials without too vigorous limitations on the plastic index. For such bases a somewhat more viscous primer than ordinarily used should be provided.

The practical importance of the two investigations which have just been described cannot be over-emphasized. A thorough study of the original work will repay the reader many times for the slight effort it requires.

PHYSICO-CHEMICAL STABILIZATION

The most important type of stabilization which belongs under this heading is the bituminous soil stabilization. Depending upon the type of soil material to be stabilized the role of the bitumen consists in either or both, supplying cohesive stability to non-cohesive materials and to reduce the water-affinity and softening of cohesive materials. In practice the term bituminous soil stabilization covers a large number of types some of which justly deserve their name, others representing borderline cases, and some obviously outside of the logical range of this classification. Table 4 enumerates the main types of bituminous stabilization and indicates the role of the bitumen employed. The underlying principles and the methods employed in the construction of these various types of bituminous soil stabilization have been expanded extensively at former meetings of this and other conferences (see Table 4) and do not warrant further treatment at this place. Recent publications on the practical phase of bituminous stabilization while being of interest to the engineer have not produced any fundamentally new aspects. At the present time the main effort of the research engineer is directed to the urgent and important problem of establishing and standardizing laboratory test procedures to predict the service behavior of bituminous soil stabilization. The underlying principle for all the test methods employed is simple and consists in the main in exposing the compacted bitumen-soil system to the effect of moisture with subsequent testing of the consistency of the system.

Table 5 gives a rudimentary outline of 4 different test methods which have been proposed for soil systems stabilized with bituminous materials. These four methods show differences in the preparation of the soil samples, and in the methods employed to mold, compact, cure and test the bitumen-soil systems. The originator of any one of these methods has silently or openly criticised those of the others. The general discussion of this subject is still wide open and a large amount of work remains to be done.

TABLE 4
MAIN TYPES OF BITUMINOUS STABILIZATION

Type of Stabilization	Bitumen Type	Bitumen Amount	Main function of bitumen	Main factor of Stabilization	Remarks and References
Sand-asphalt	RC, MC & SC, cutb., emulsions, Tars, fluxes & powdered asphalt	3-8%	supplying cohesion	Cohesion plus friction	Symp. on Stabilization Proc. 11th National Asphalt Conference, 1937. pp. 28-69. Proc. Mont. N. Bit. Conf. 1938, pp. 311-320.
Soil: more than 65% passing No. 40 sieve	MC & SC, cutbacks, emulsions, Tars	7-8%	water-profing	Cohesion	Symp. on Stabilization as above. Also report from Mo. State Highway Dept.
Graded soil aggregate systems	Mainly emulsions but other types of bitumen also	2-5%	water-proofing (cohesion)	Friction plus interlocking	See especially reports on stabilization by means of emulsions.
Oil-gravel	SC-2 SC-3 TM-2	1½-3%	water-proofing and lubricating	Friction plus interlocking	Proc. Mont. N. Bit. Conf. 1938, pp. 387-400.

It appears that the fundamental differences in the proposed tests lie in the methods of curing and water exposure of the speciments rather than in the type of stability test. The proposed curing methods are:

1. Air drying for seven days.
2. Air drying with consecutive oven drying to constant weight at 140° F.
3. Air drying with consecutive oven drying to constant weight at 160° F.
4. No curing.

It is a well known fact that the drying out of a moist bitumen-soil system may change irreversibly its future susceptibility to the effect of water; it is also known that in certain climates a thorough drying of the stabilized road will take place while under other skies no water loss will occur at all. It is obvious that drying at moderate temperatures and no curing will take care of these two extremes. The main problem seems to be the proper type of curing for the intermediate cases where moderate drying takes place in the road. Uniform partial drying can be achieved by permitting the samples to come to equilibrium in closed systems with definite vapor pressures, but this procedure is too time consuming to be recommended for a standard test. For practical purposes it might be well to test the specimens at both the uncured and the cured condition and hope that the actual behavior lies in between. Unfortunately, experiments in the authors laboratory on samples cured over sulfuric acid solutions of different concentrations showed rather unpredictable inclinations to softening and water intake. In view of the good record of the test No. 1 which provides for oven drying at 140° F. even when such drying can not be actually expected on the road it might be worthwhile to look for an explanation. An investigation on the physico-chemical factors of importance in bituminous stablization (4) showed that the behavior of air dried soil-

TABLE 5
COMPARISON OF THE DIFFERENT METHODS USED IN TESTING BITUMEN-STABILIZED SOIL SYSTEMS

Preparation	Molding and Compacting	Curing	Exposure to Water	Testing for Stability	Remarks
1. Soil and aggregate air dried and screened. Material passing ¼" sieve is employed.	Water added to reach range between P.L. & L.L. thoroughly mixed with bitumen, dried until it can be compressed under 3000 lb. load. test cylinders 2"x4".	Air drying in laboratory and drying to constant weight in oven at 140° F.	Exposure in capillary water. Sides of sample wrapped with cellophane. Test temperature 70°-80° F. Absorption time 7 days.	Extrusian test: Wet end downward. Punching shear test.	Proposed to A.S.T.M. committee D-18, Subcommittee V. as "Method of test for stabilization with emulsified asphalt." See there for further detail.
2. Soil dried at 220° F. pulverized screened Material passing No. 4 sieve is employed.	Water and cutback added in ratio 4:6 the total amount controlled by the exudation point. Tamped in H.F. 2" diameter mold compressed under 10,000 lb. load. Cylinders 2"x1".	4 specimens A. dried at 140° F. B. dried at 140° F. C. uncured. D. uncured.	Specimens B and D half immersed in water for 7 days. Water absorption and swell determined.	Hubbard-Field Stability on all 4 specimens with emphasis on C and D.	Soil Stabilization using asphalt cutbacks as binders. Proc. Ass. Asphalt P. Techn. Vol. 10, 1-36.
3.	Water and emulsion to approximate liquid limit of soil. 24 hours curing under damp cloth. Compaction under 300 lb. load. Cylinders 4" diameter, 1.12" height.	Air drying in laboratory and drying to constant weight in oven at 160° F.	Exposure to capillary water for 24 hours, and reweighed.	Punching shear test with special apparatus.	The stabilization of soils with emulsified asphalt. ibidem. Vol. 10, 37-85. Note especially discussion by McKesson.
4.	Wet mixing with bitumen. Molded into small blocks.	Air drying for 7 days	Submersion in water for 7 days.	Visual and Manual inspection.	ibidem. Vol. 9. p. 289.

bitumen systems is more closely related to the chemical composition of the soil materials than to their plastic index. On the other hand, specimens taken out of a stabilization job after a year in service showed that the moisture content and stability went parallel to the plastic index of the original soil. It is quite conceivable that air drying of the specimens is permissible in the laboratory testing of such mixtures the soil binder of which possesses a very small or zero plastic index even if the stabilized base never experiences thorough drying. For soil systems containing a relatively large amount of soil binder possessing an appreciable plastic index water exposure without preceeding curing appears to be preferable.

What procedure should be used for the water absorption test? Short time submersion of the test specimens has the disadvantage of trapping air in the treated specimens which resists the establishment of equilibrium conditions. Long time submersions (36 days) which give good results are too time consuming. Both partial immersion or exposure to capillary water appear to be practicable and deserve recommendation. Reduction of the air pressure in the absorption cabinet may permit the use of shorter exposures than are necessary at the present time.

Very interesting and very valuable work toward standardization of these testing procedures is being done by the respective subcommittees of the A.S.T.M., and its results should go a far way in rationalizing and standardizing bituminous soil stabilization.

A report on the "Physico-chemical factors of importance in bituminous soil stabilization" by the Missouri State Highway Department has just become available. This report attempts to correlate the results of bituminous stabilization with the surface-chemical properties of the soils and of the bituminous materials employed. Although the study is restricted to such systems in which thorough drying at room temperature has taken place and claims exact validity only for such conditions, valuable extrapolations can be made from the accumulated data, especially those on the soil properties, which should be of use in judging the behavior of systems which have not been dried to the same extent. General results of this study are:

1. The water resistance of bitumen-stabilized soils depends on the amount and chemical character of the fine soil constituents, the chemical character being expressed by the silica sesquioxide ratio of the clay and colloid particles and by the amount and kind of exchange ions. The smaller the ratio and the higher the valency of the exchange ions, the better is the suitability of the soil.
2. The water resistance of bitumen stabilized soils depends upon the type of bitumen used. According to their efficiency the bitumens can be divided into three groups:
   - (a) Best—positive asphaltic cutbacks and high-temperature tars;
   - (b) Intermediate—standard negative bituminous materials.
   - (c) Poor—Low-temperature tars.

The results obtained on sodium and potassium saturated soils seem to explain the failures of bituminous treatments occurring in our alkali regions. If bituminous treatments are desired in alkali soils furfural-type activators which resinify in the presence of alkali can be warmly recommended.

Chemical Stabilization:

The most important type of chemical stabilization is the hardening of soils by means of Portland Cement. The development of the basic principles for producing consistent predictable results in processing soil cement mixtures for light traffic roads is due almost entirely to the activities of the Portland Cement Association. Of course, for the practical application of this process invaluable contributions have been made by the different highway departments which cooperated with the P.C.A. The laboratory tests, which have been proposed by this organization are logical and severe. They include determinations of—

1. Proctor compaction curves,
2. Resistance to wetting and drying cycles,
3. Resistance to freezing and thawing.

The tests themselves are well known to the highway engineering fraternity and need not be described in detail at this place.

It is very important that the results of soil hardening could be connected to a certain extent with the soil grouping of the B.P.R. and with the soil tests which are the basis of this grouping. According to Sheets and Catton (5) effective hardening can be achieved with soils of the following properties:

1. Liquid limit below 50.
2. Plasticity index below 25.
3. Clay content below 35.
4. Percentage of solids at maximum density 60 or greater.
5. Soil possesses a ''regular'' moisture density curve.

Since all these 5 properties of natural soils can be changed by ionic treatment it can be foreseen that a combination of ionic treatment with regular cement hardening will be one line of future development in this field. Preliminary experiments executed at the Missouri State Highway Department on the effect of ionic pretreatment of Cecil soil before hardening with Portland Cement revealed a great influence of such pretreatment. Still greater effects can be expected with podsolic soils which are more conducive to change by ionic treatments than the lateritic Cecil clay.

This brings up the whole question of the correlation of the surface chemical properties of the soils with their susceptibility to hardening by means of Portland Cement. In the case of bituminous stabilization it has been shown that the effect of the surface chemical properties of the soils on the quality of stabilization is more direct than that on their own consistency properties. Correspondingly, it can be expected that the chemical classification of soils will also be of great value for the further development of soil hardening. The large amount of available information on the chemistry of the American soils promises easy advances in this direction.

No fundamental advances have been recently reported in the other types of chemical soil stabilization. Therefore, no additional information appears to be necessary over that given and indicated last year.

SUMMARY

This report contains a rational classification of the different types of soil stabilization. Recent advances and the present status of the practically most important type of each class have been sketched and the probable future direction of some of them has been outlined. To resume the allegory of the introduction it is hoped for the best of soil stabilization that neither she herself nor her present tutors forget her humble origin or her inherited potentialities.

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Factors in the Hardening of Hot-Mix Asphaltic Materials

By

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The hardening of hot-mixed paving mixtures during preparation and placing is due primarily to oxidation of the asphalt cement by air at the elevated temperatures employed. The variables in this oxidation which are subject to control are the oxidation characteristics of the asphalt, the temperature of the mixture during the mixing and placing, and the time for which the mixture remains at these elevated temperatures.

The important oxidation characteristics of the asphalt are the rate of oxidation at mixing temperatures, the variation of this rate with temperature, and the extent of the change in the other properties of the asphalt for a given degree of hardening. That there are substantial differences between asphalts in these respects is beginning to be known by paving technologists but is not as yet generally appreciated. Figure 1 shows results obtained on several types of commercial paving asphalts, using a laboratory oxidation test developed by the State Highway Department of Michigan[1]. This figure shows the times required in this test to harden the asphalts from the original penetrations to 30 penetration the value which Hubbard and Gollomb[2] concluded was the lower limit for the penetration of the asphalt in the pavement if difficulty from cracking is to be avoided. In addition to the obvious differences between asphalts of the same type but different original penetrations, it is apparent that there are substantial differences between the different types. Thus, a 100 penetration asphalt of type D oxidized to 30 penetration in less time than was required for a 60 penetration asphalt of type A, and over the range from 60 to 100 penetration type D asphalts oxidized 50 to 100% faster than type A. The advantage to be gained by using the latter type rather than the former is obvious.

The time required to produce a given amount of hardening by oxidation in the above test, or similar ones, is many times that required in the actual mixing operation. At first glance this might be considered as indicating that the hardening in the latter case is caused, in part, by other factors than oxidation; in particular, by evaporation of volatile material from the asphalt. Present day asphalt cements, which usually are produced by pipe still distillation, contain very little material which is volatile at mixing temperatures, and it seems improbable that much of the hardening in mixing is due to evaporation. The substantially higher rate of hardening in the actual mixing operation appears rather to be due to the enormously greater exposure of the asphalt, distributed over the aggregate in thin films, to the action of air. It is also possible that some aggregates contain minerals which catalyze this oxidation. To the writer's knowledge this possibility has never been investigated.

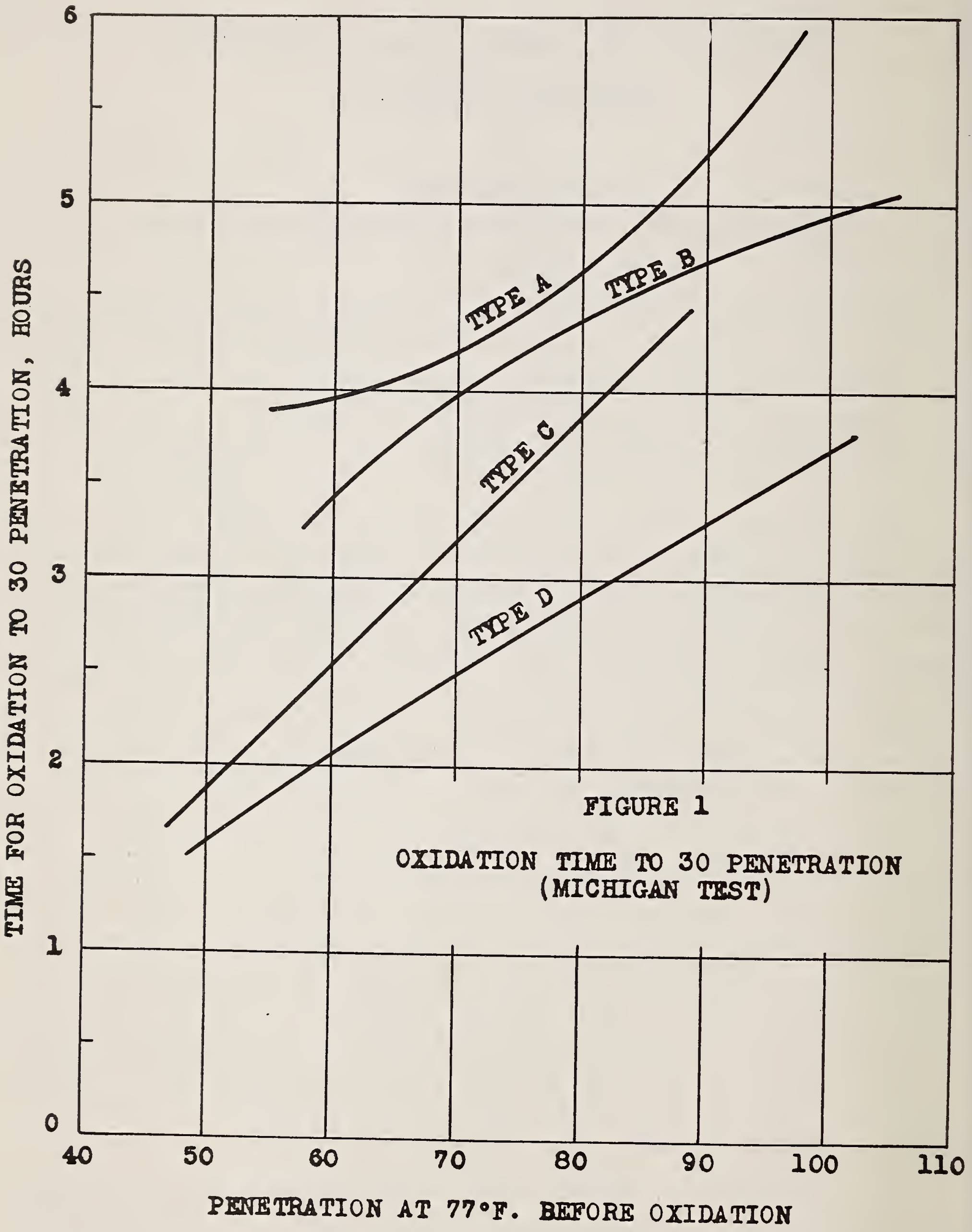

FIGURE 1

OXIDATION TIME TO 30 PENETRATION
(MICHIGAN TEST)

Another important difference between asphalts is in the extent to which their properties other than penetration are changed by oxidation. For example, with the types of asphalt cements studied to date by our laboratory the ductility after oxidation from 100 penetration to 30 penetration in the above test usually is inversely related to the time required for that oxidation. That is, those products which to the greatest degree retain ductility after hardening by oxidation will in general oxidize the most rapidly. It would therefore appear unwise to require that the asphalt cement have unduly high ductility after subjection to oxidation, since this appears to be associated with low resistance to hardening.

Asphalts from different crudes, or prepared by different processing methods may differ appreciably in viscosity at mixing and placing temperatures, and these temperatures should be adjusted to the viscosity characteristics of the asphalt being used. Figure 2 shows the viscosities, over the temperature range from 300° to 400° F., of asphalts of 57 penetration at 77° F. of the types A, B and D shown in Figure. 1. It will be noted that at any given temperature the viscosity of the type D asphalt is approximately twice that of the type A, or the same as that of the type A at a temperature 25 to 30° lower. Since distribution of the asphalt over the aggregate surface in the mixing operation depends upon the viscosity of the asphalt at the mixing temperature, asphalts of relatively low viscosity permit the use of correspondingly lower mixing temperatures, with the important attendant advantage of a lower rate of oxidation during mixing. In this connection also, the addition of a small quantity of flux oil to the mixture, to reduce the viscosity of the asphalt, has given satisfactory results when carefully done and offers

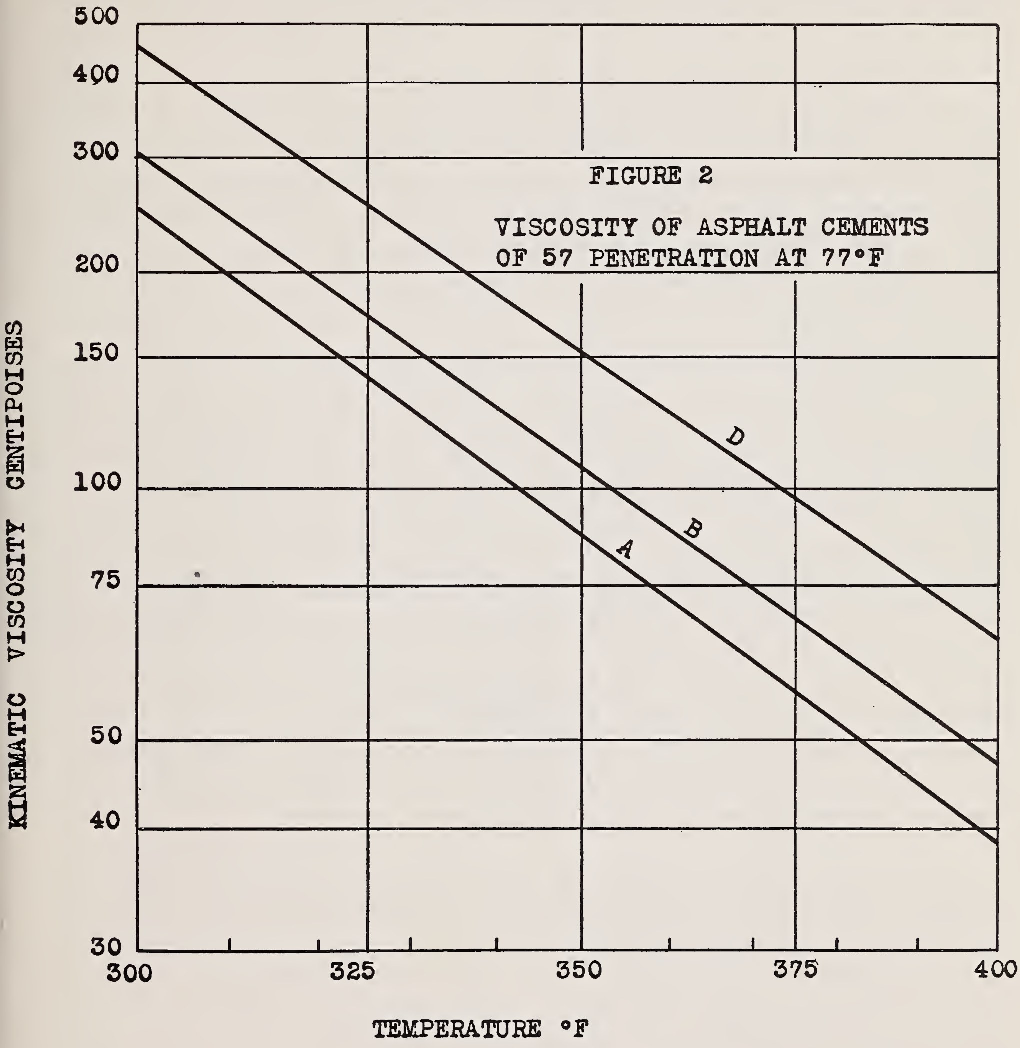
FIGURE 2

VISCOSITY OF ASPHALT CEMENTS OF 57 PENETRATION AT 77°F another method of avoiding high mixing temperature. The possibilities of reducing the hardening during mixing and placing by lowering the mixing temperatures in these ways have not received much attention from workers in this field, and apparently are not generally appreciated.

The third variable in oxidation during mixing and placing is the time and mixture remains at elevated temperatures. Control of this time depends upon the mechanical equipment used and the scheduling of the operations, matters which are outside the scope of this paper.

Practically all organic substances are oxidized by air at atmospheric temperatures, and asphalts are not immune to this action. In addition, they are also subject to a phenomenon known as ''age hardening,'' which apparently is not caused by oxidation. The deterioration of paving mixtures after placing from chemical or physical changes within the mixture appears to be due either to hardening of the asphalt cement in the above ways, or to displacement of the asphalt from the surface of the aggregate by water, i. e., failure of the asphalt-aggregate bond.

This deterioration has sometimes been referred to as ''weathering,'' a term in general use to denote those changes occurring in paints, asphalt roof coatings and other materials which are directly exposed to the combined action of sunlight, air, water and fluctuating temperatures. Application of this term to the changes occurring in pavements is rather unfortunate, as it has led to some apparent confusion as to the factors causing deterioration. The asphalt in the bulk of a pavement is not exposed to the sunlight or strong artificial light, or to the leaching action of frequently repeated water sprays or rain which carry away water-soluble oxidation products, which are the most important factors in weathering tests intended for paints or bituminous coatings. It is, therefore, not surprising that accelerated weathering tests intended for use on the latter types of material have not given results on paving asphalts or paving mixtures which correlate with actual service behavior.

Of the two types of hardening which occur in pavements under service conditions, that due to oxidation will be determined by the tendency of the asphalt to oxidize under these conditions and by the extent to which air has access to the asphalt. This type can obviously be reduced, although never eliminated entirely, by using asphalts which oxidize slowly and mixtures designed to have the minimum voids consistent with satisfactory behavior in other respects. The other type, so-called age hardening, is not an oxidation phenomenon, as shown by the fact that careful reheating of age-hardened asphalt will return it to its original consistency. This type of hardening proceeds most rapidly immediately after the asphalt has solidified, the rate of hardening diminishing rapidly with time, and is probably due to changes in the colloidal or internal structure of the asphalt. Mechanical working has the same effect as heating, although to a somewhat lesser degree, which may account for the observation that pavements not subjected to traffic in many cases deteriorate more rapidly than those which are in use.

Very little work has been published on the relationship of hardening in service to structural deterioration of the pavement, or the extent to which such hardening may occur before deterioration becomes serious. Hubbard and Gollomb[2] concluded that in the case of sheet asphalt pavements penetrations below 20 were conducive to cracking, and that serious difficulty from cracking would not occur if the penetration could be kept above 30. On the other hand, instances have been reported[3] of pavements in excellent condition after 25 years, the asphalts in which had penetrations of less than 20. Further research on this point, to establish the safe lower limit for the asphalt in the pavement, is badly needed.

Many of the aggregates used in asphalt pavements are preferentially wetted by water rather than by asphalt, and rupture of the bond between the asphalt and the aggregate, by water which has entered the pavement after placing, probably is a more important cause of trouble than is generally appreciated since this may occur without visible displacement of the asphalt from the aggregate surface. Strictly speaking, this is not deterioration of the materials themselves in any way—the asphalt and the aggregate are the same as before. But there can be no question that from the standpoint of the pavement as a structure it represents serious damage. Considerable research has been done on the adhesion of asphalts to aggregates in the presence of water, and tests are now available by which materials having particularly poor behavior in this respect can be detected and their use avoided. Work along more fundamental lines, aimed at direct control of the surface or interfacial forces which determine adhesion, is in progress and has given definite promise of leading to practical methods of ensuring a permanent bond between the asphalt and aggregate which will not be destroyed by water.

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Cracked vs. Uncracked Road Oils

By

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Coincident with the expansion of the automobile industry and the corresponding increase in the consumption of motor fuels, a refining practice has been developed whereby the heavier petroleum products of higher boiling points can be converted into the more desirable volatile products which find a readier and wider range of commercial application. This process is known as "cracking," which is commonly referred to as "pressure distillation."

The principles of "cracking" usually involve distilling under pressures above atmospheric. This results in a decomposition of the heavier oils, with the resultant formation of a large percentage of products of low gravity and boiling points. The relative importance of this phase of refining can be understood readily when it is considered that well over 50% of the gasoline manufactured today is produced by various cracking processes.

While the advent of cracking methods has proven a boon to refiners in the greatly increased production of light oils and motor fuels, it has also created a problem where the residues of such distillations are concerned.

In some cracking processes, the residual oil is re-distilled to such an extent that the final residue is chiefly coke. However, in most processes of pressure distillation the final residual products are liquid or semi-solid resdues which are, physically, very similar to the residues of non-cracking or straight run distillations.

Since these distillation residues are the basis of most of the road oils and asphalts used, there has been on the market for some years two distinct types of bituminous road materials, i. e.—cracked and uncracked.

Although these two types of bituminous products can be manufactured so that their physical characteristics are similar, they are greatly dissimilar in their chemical constitution. It has also become apparent, after several years of use and experimentation, that there is a great dissimilarity in the road making qualities of cracked and uncracked oil.

Cracked road oils and asphalts have been used in road building in practically every section of the country. In some cases the reports have been that the use of this material has been fairly satisfactory. However, in by far the majority of cases, the reports on the use of cracked oil have been decidedly adverse.

The reports of satisfactory use of cracked oil have come mainly from the middle Western States and some of the Northern States. The fact that some success has been found in the use of cracked material in these sections is due primarily to the climatic conditions in those regions. Cracked oils do resist or shed water. This is because of the heavy specific gravity of the material. Cracked oils are almost invariably heavier than water.

The use of cracked bituminous materials has been most unsatisfactory, however, in the more or less arid regions of the Southwest. New Mexico, in particular, has had some sad experiences with cracked road oils. Because of this, a series of laboratory experiments was begun some time ago in the laboratory of the New Mexico State Mighway Department in an endeavor to show the comparative qualities of cracked and uncracked road oils under identical conditions.

A large quantity of well graded sand was prepared, all of which passed the No. 10 sieve. This sand was well mixed, the exact grading ascertained, and the amount of oil necessary for the correct mixture was figured from the New Mexico Formula.

Eight samples of road oil from various sections of the country, and of approximately the same consistency, were secured. Four of the samples were uncracked oil and four were cracked oil. Characteristics of these road oils are listed in Table 1.

Equal amounts of the sand were taken and the formula amount of oil from each sample was added and thoroughly mixed. Approximately 100 briquettes 1″ by 1″ in size were molded from each of these mixtures at a compressive strength of 650 kilograms. These eight hundred briquettes were all stored in the laboratory in the same room.

At the same time, other equal amounts of the sand were taken, the formula amount of oil from each sample added, and all mixed in exactly the same manner as the first batch. These eight mixtures were placed in large flat pans and set outside the laboratory to be exposed to the weather.

On the days set for breaking, ten of the briquettes from each batch already made up were broken under compression and the average strengths obtained. On the same days, the weathered material was brought into the laboratory and ten briquettes, 1″ by 1″ in size, were molded from each batch under 650 kilograms pressure. These briquettes were broken immediately in the same manner as the pre-molded briquettes and the average strength under compression recorded.

In order to break these briquettes in the same manner, it was necessary to use special apparatus. A machine was devised by the Testing Engineer for this purpose. It consisted chiefly of a vertical plunger, with a head about two inches in diameter, to which a steady flow of lead shot could be applied. This weight of shot was applied to the top of the plunger until the briquette collapsed, at which time the flow of shot was automatically cut off. The exact weight necessary to break the samples was thus ascertained to within 5 grams.

These briquettes were broken in this manner at various times over a period of three years. All samples were broken at approximately the same temperature, 75° to 80° F. Everything possible was done to make all test conditions identical and all measurements exact so that the only variable factor in the experiment would be the oil used.

The compressive strengths of these samples throughout the three year period are shown, for the pre-molded briquettes, in Table 2. The strength of the briquettes molded from weathered material are shown in Table 3.

All these results are shown graphically in Figures 1 to 8.

SAMPLE NO. 1—UNCRACKED OIL

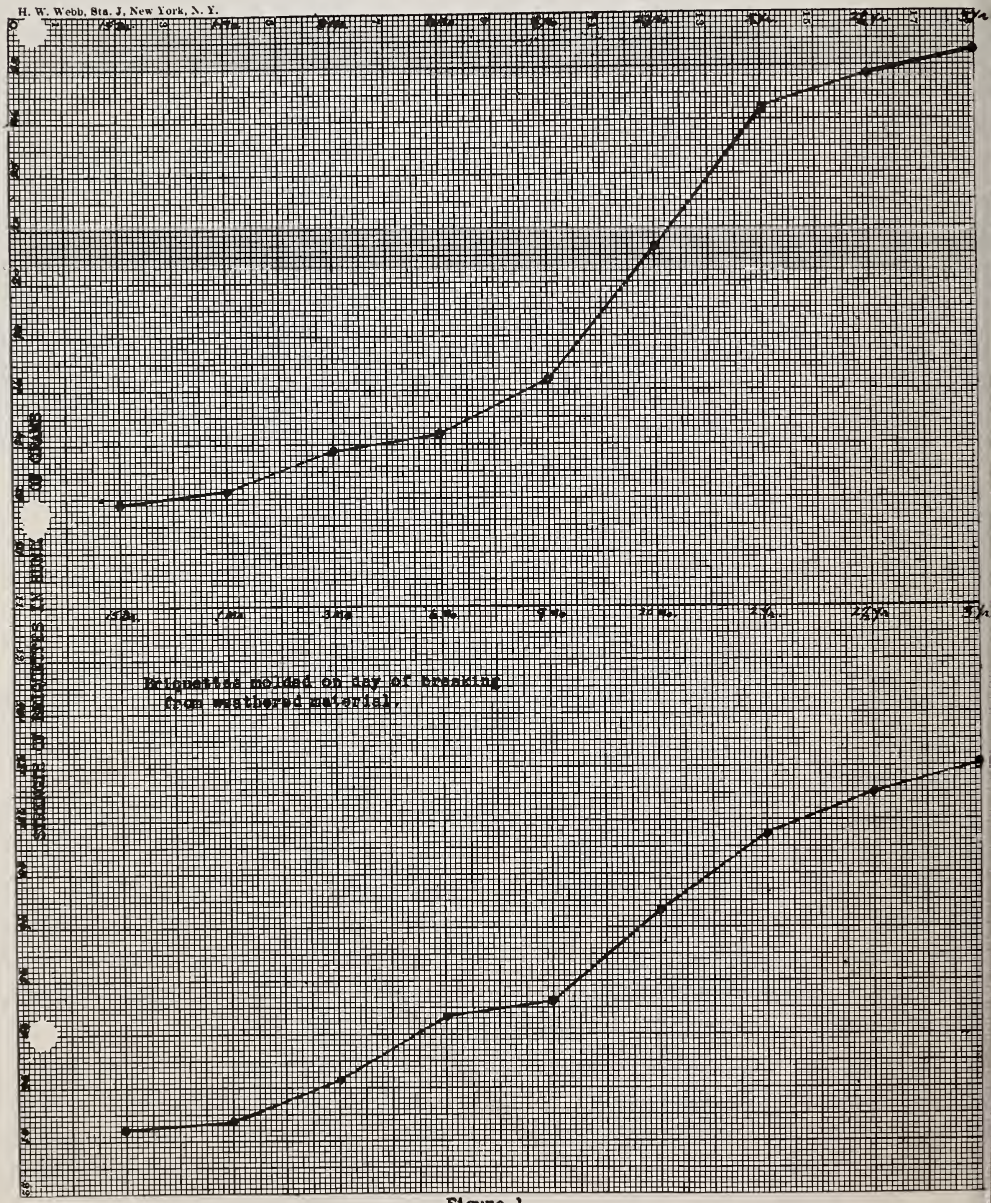

Figure 1

Sample No. 2 -- Uncracked Oil

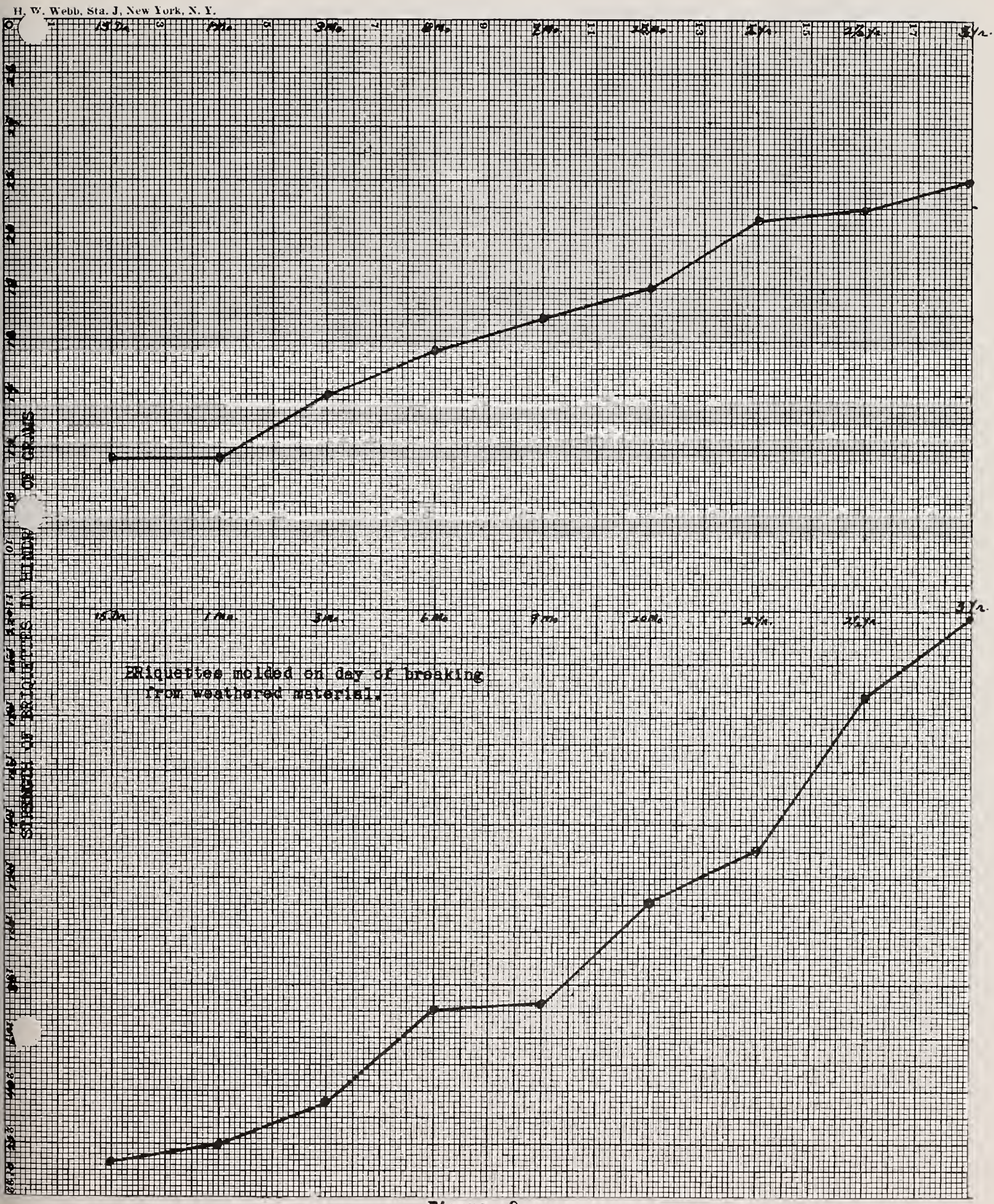

Figure 2

SAMPLE No. 3 - - UNCRACKED OIL

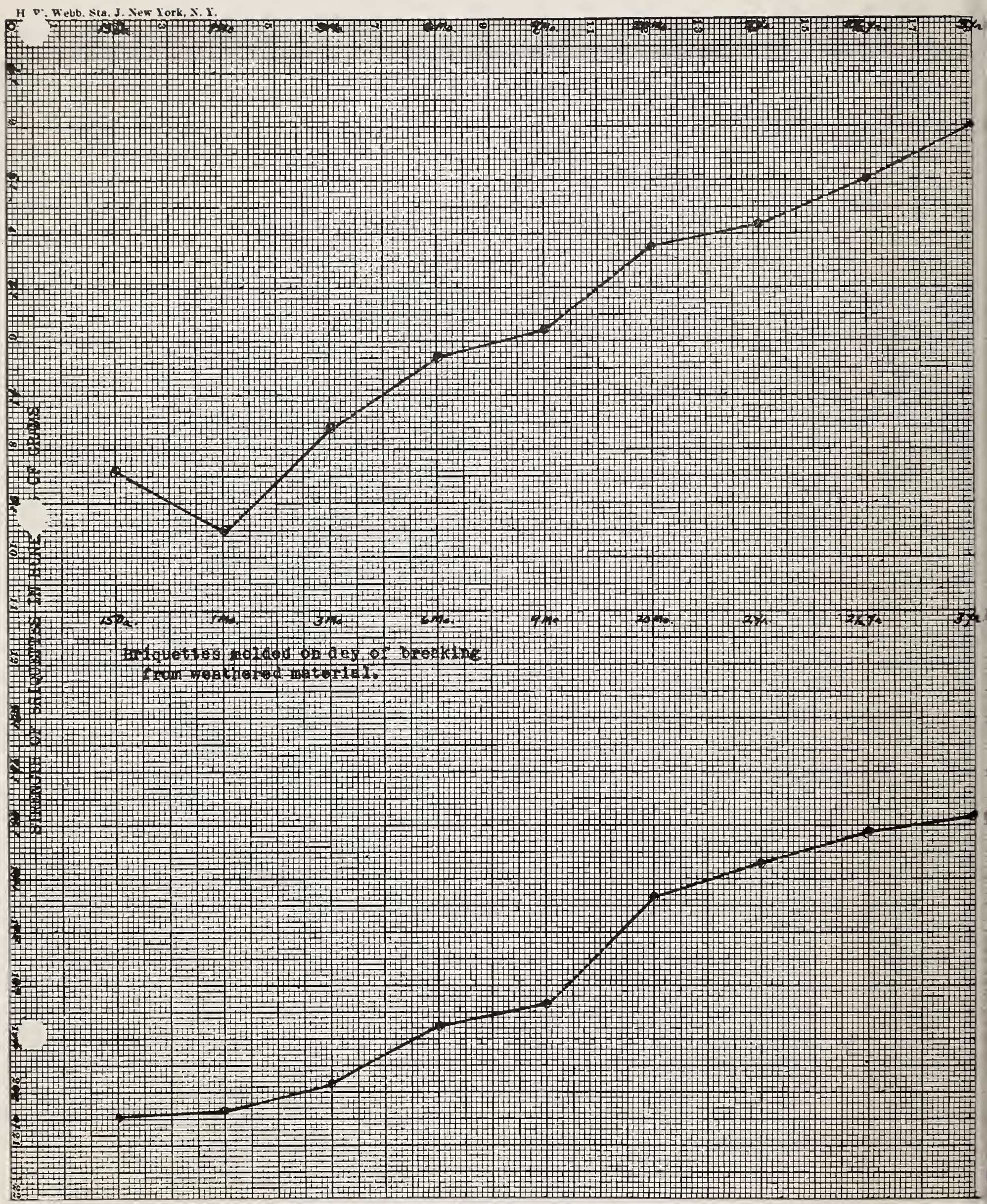

Figure - 3

SAMPLE No. 4 - - - Uncracked Oil

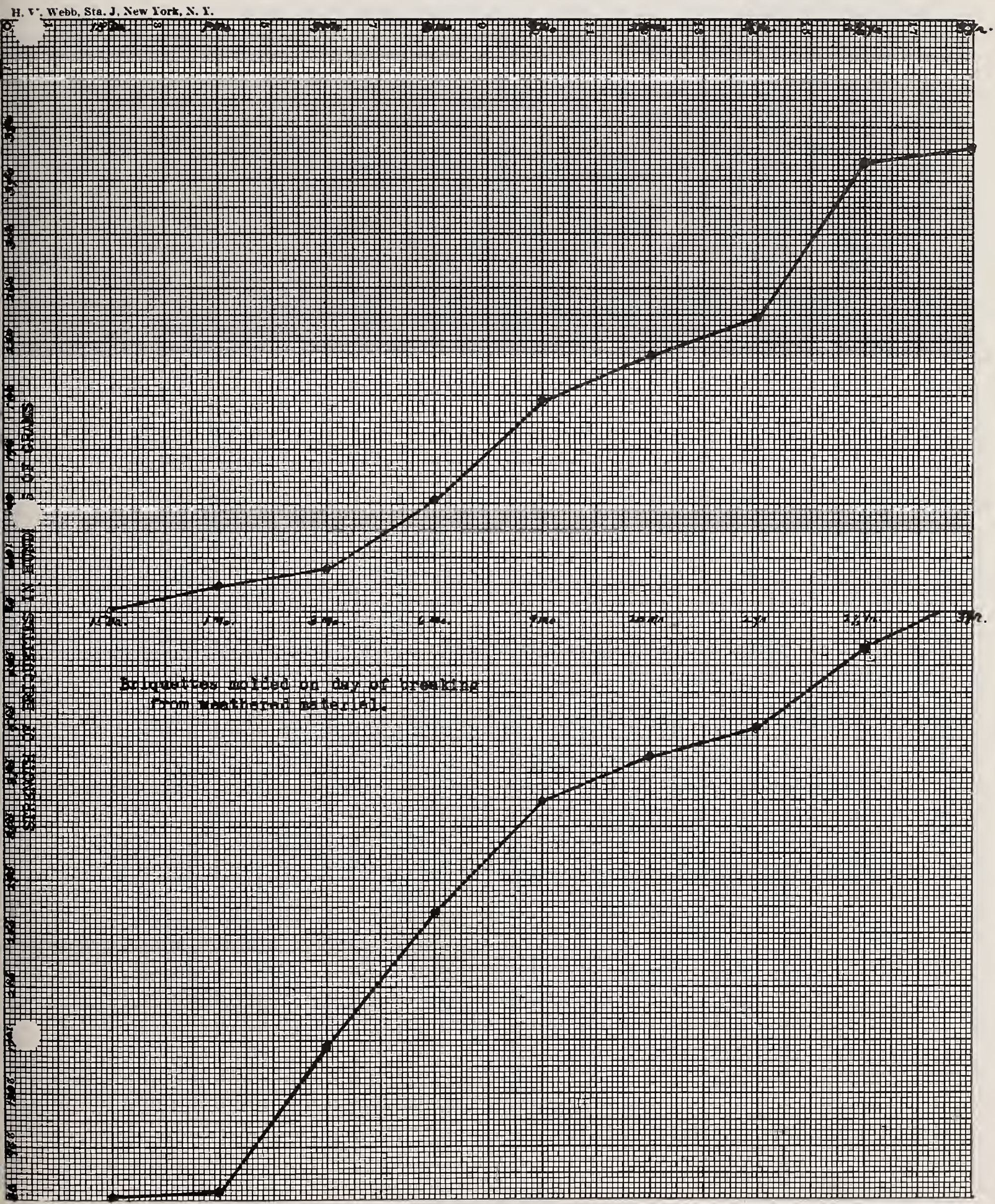

Figure - 4

SAMPLE No. 5 - - - - CRACKED OIL

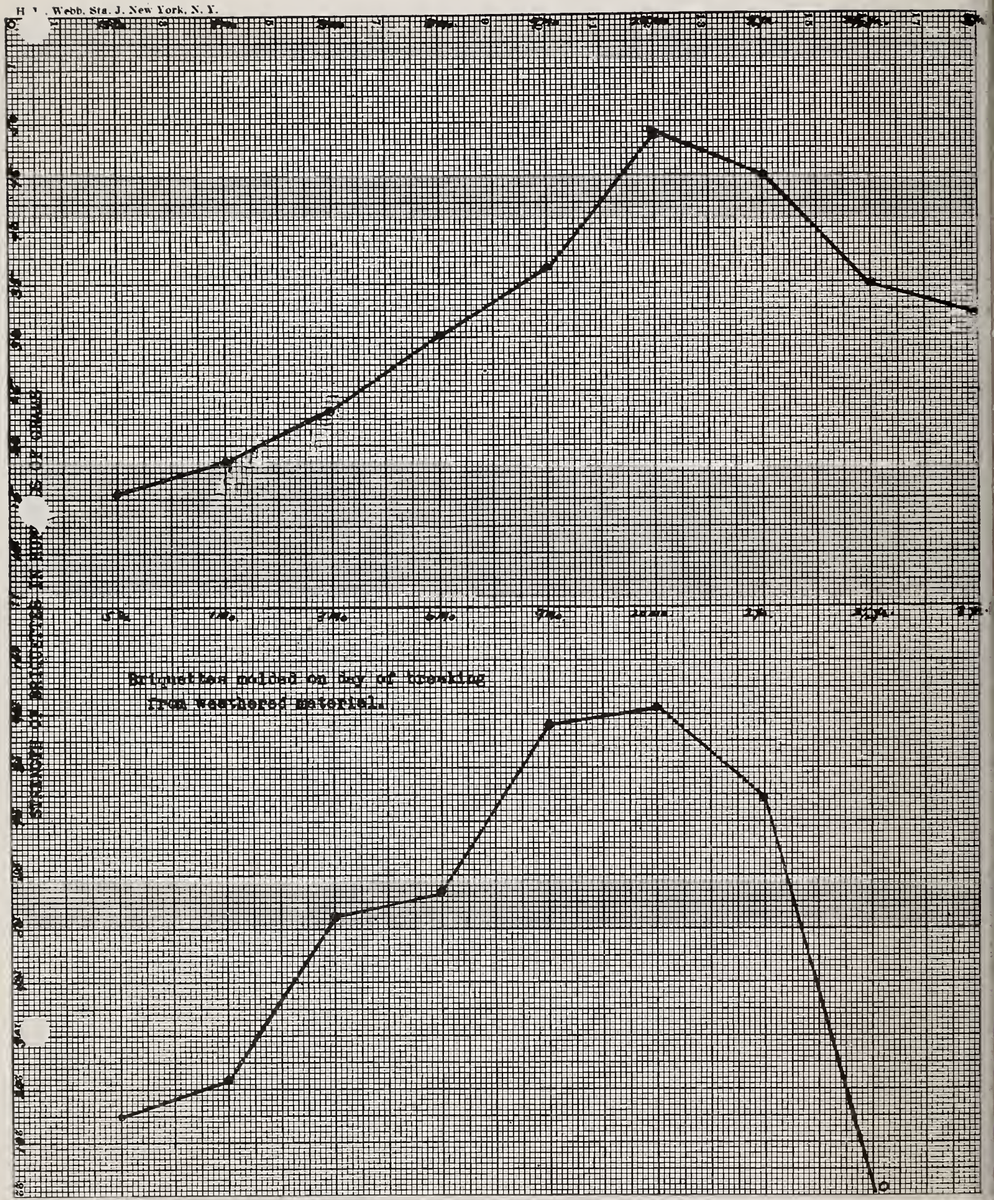

Figure - 5

SAMPLE No. 6 - - -CRACKED OIL

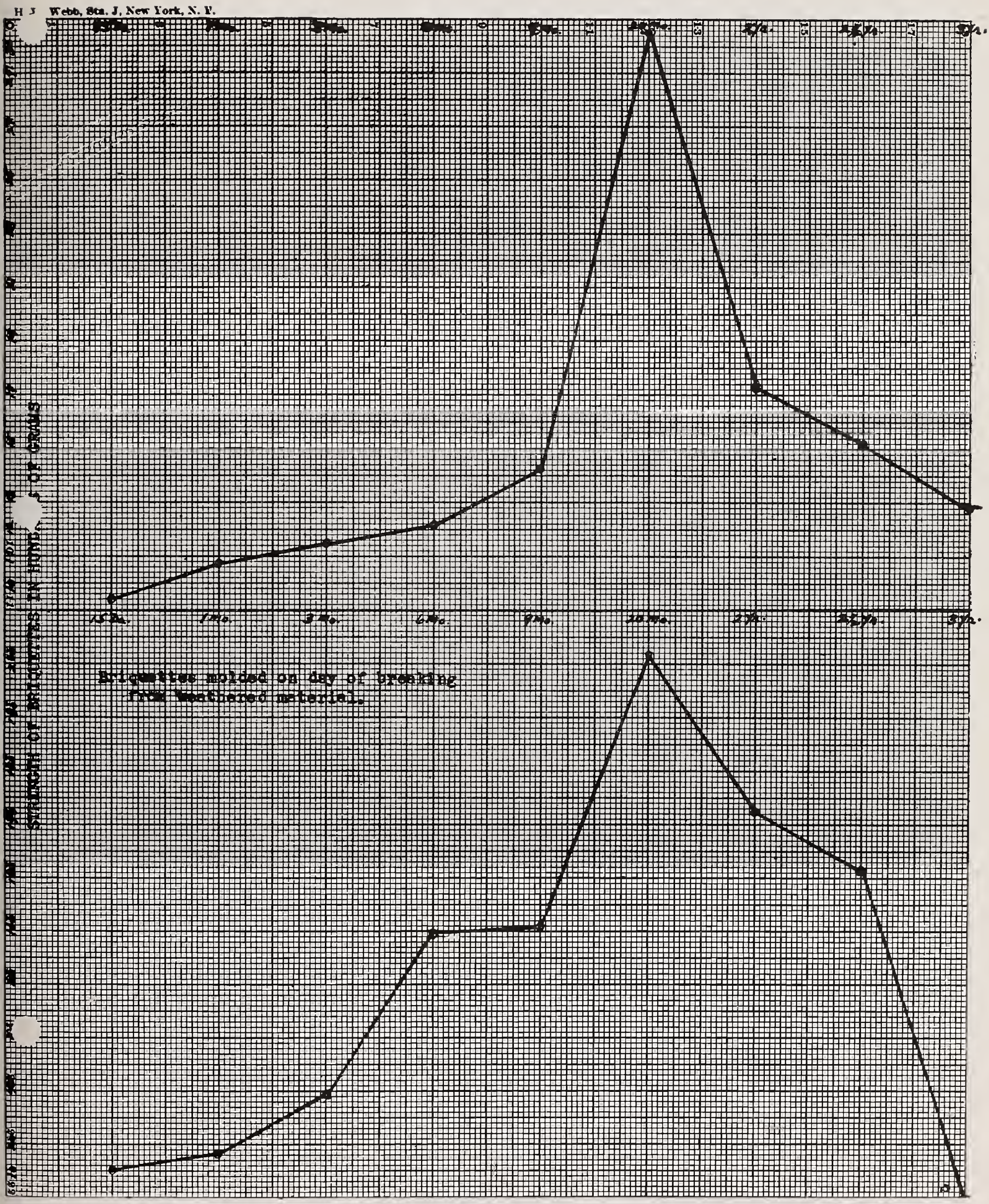

Figure - 6

SAMPLE No. 7 - - - CRACKED OIL

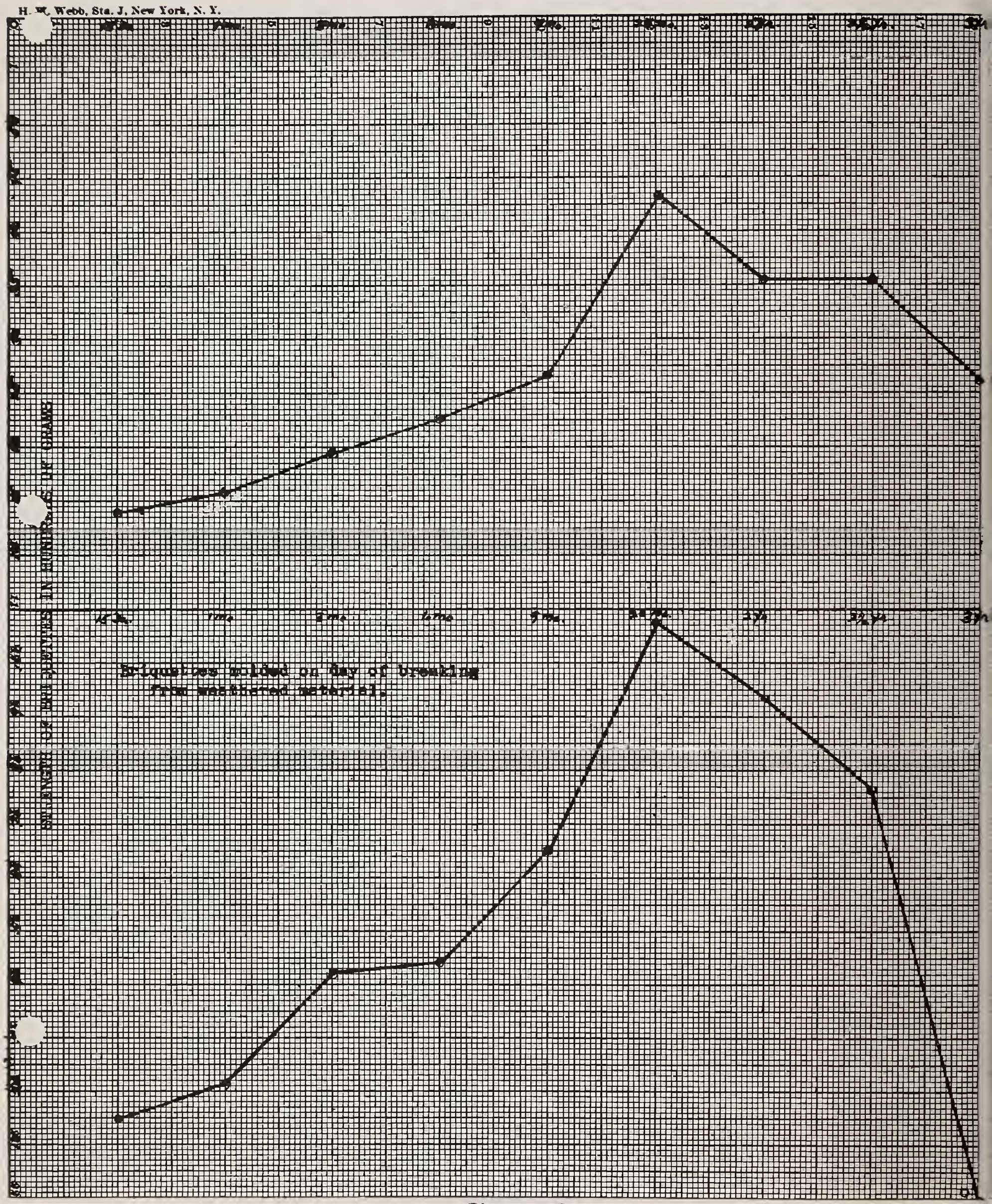

Figure - 7

SAMPLE No. 8 - - - CRACKED OIL

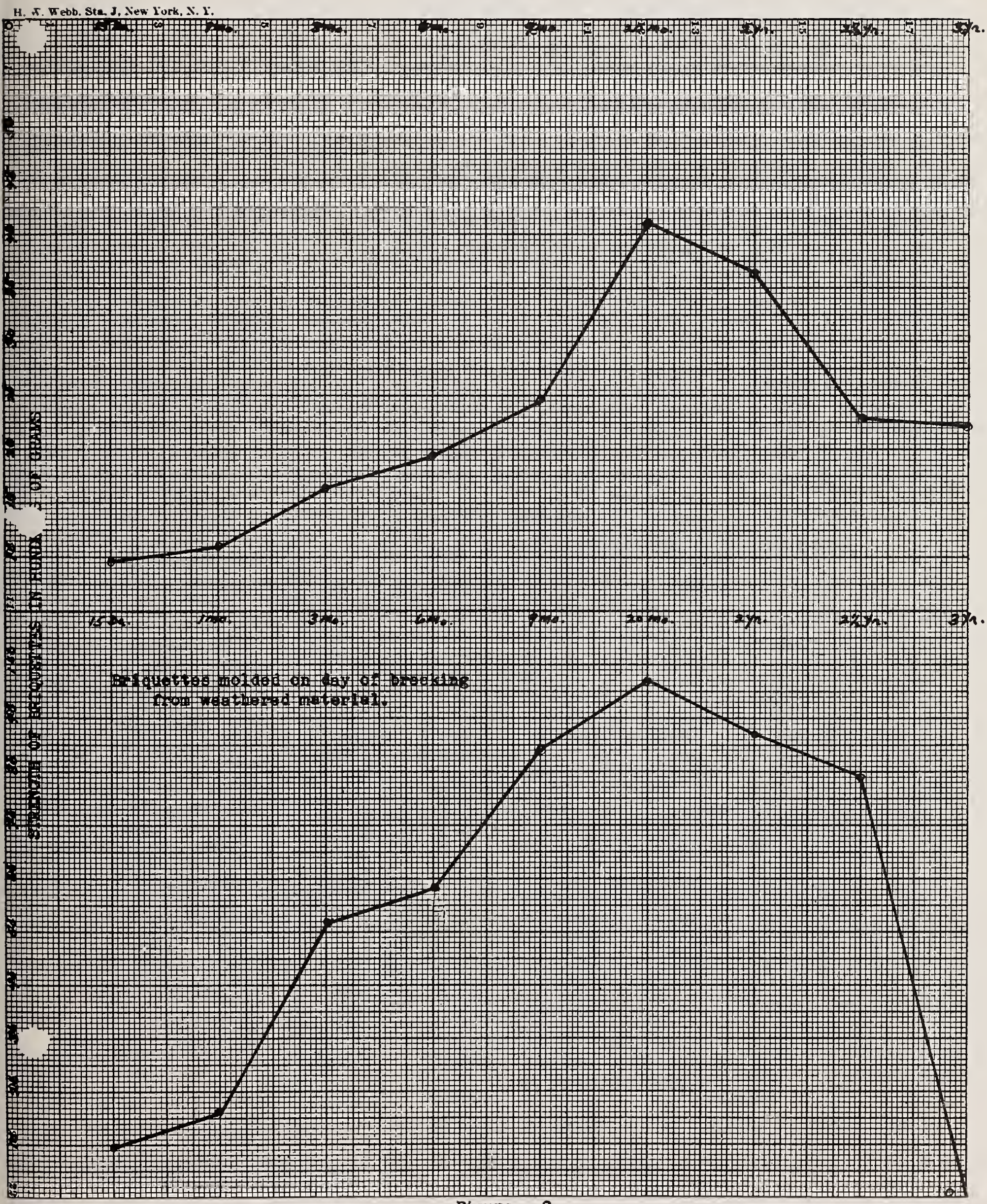

Figure - 8

TABLE NO. 1
CHARACTERISTICS OF OIL SAMPLES USED

Sample No.	Source	Type	Grade	Sp. Gravity
1	Oklahoma	Uncracked	SC-4	.9888
2	California	Uncracked	SC-4	.9923
3	New Mexico	Uncracked	SC-4	.9813
4	California	Uncracked	MC-4	.9779
5	Kansas	Cracked	SC-4	1.056
6	Texas	Cracked	SC-4	1.099
7	Wyoming	Cracked	SC-4	1.035
8	Missouri	Cracked	SC-4	1.062

TABLE NO. 2
COMPRESSIVE STRENGTH OF PRE-MOLDED BRIQUETTES (in grams)

No.	15 days	1 Mo.	3 Mo.	6 Mo.	9 Mo.	20 Mo.	2 Yrs.	2½ Yrs.	3 Yrs.
1	1187	1230	1378	1437	1637	2134	2647	2766	2844
2	1167	1170	1404	1568	1687	1805	2057	2090	2208
3	1032	972	1070	1136	1159	1239	1259	1302	1350
4	2099	4031	5100	10330	17625	21022	23725	35420	36390
5	1546	1837	2336	3020	3657	4904	4494	3490	3254
6	940	1075	1148	1219	1419	3088	1739	1524	1272
7	1398	1575	1946	2280	2673	4315	3558	3566	2608
8	988	1110	1644	1939	2446	4118	3630	2276	2220

TABLE NO. 3
COMPRESSIVE STRENGTH OF BRIQUETTES MADE FROM WEATHERED MATERIAL

No.	15 days	1 Mo.	3 Mo.	6 Mo.	9 Mo.	20 Mo.	2 Yrs.	2½ Yrs.	3 Yrs.
1	1459	1735	3341	5732	6223	9661	12550	14040	15118
2	1351	2026	3575	7096	7226	11517	13130	18810	21797
3	1076	1322	2278	4482	5291	9368	10548	11800	12380
4	3393	3678	16540	26680	35000	38262	40324	46320	50000+
5	1475	2208	5246	5666	8803	9912	7440	0	0
6	1063	1619	3860	9889	10105	20380	14586	12350	0
7	1497	2149	4256	4393	6509	10746	9352	7640	0
8	993	1628	5182	5779	8419	9689	8719	7883	0

An analysis of these results indicates clearly the inferiority of the cracked road oil, not only under ordinary curing conditions, but also under the most extreme weathering conditions. It is significant that in every case the cracked oil reached the ultimate in its binding power in approximately 20 months, while the uncracked oil was, also in every case, increasing in its efficiency as a bonding medium up to the time of the termination of the experiments.

The results of these experiments also corroborate conclusions which were drawn some time ago in this State as to the feasability of using any cracked material in bituminous road-building. Costly experience with the use of cracked road oil and its extremely poor weather-resisting properties could lead only to the conclusion that such material had no place in road building, at least in the Southwest.

There may be some places where the use of cracked oil may be desired but it is our observation that the trend is toward the elimination of such products in all sections of the country.

The question arises of how to write specifications to eliminate cracked, and otherwise injured, road oil or asphalt. It is a fact that under most ordinary specifications, or specifications designed to cover only the grading and physical requirements of an oil, it is possible by clever manufacture to make a cracked oil which will meet all the requirements of the specifications.

Several so-called quality tests have been advanced in the last few years for use in tightening specifications against poor quality road oils. The test which, from observation, seems to have been most widely adopted, or at least tried out, for this purpose is the Oliensis Spot test.

In the laboratory of the New Mexico Highway Department we have been using and experimenting with the spot test ever since it was brought to the attention of the country in 1933. Our observations in the use of this test have led to the conclusion that, while this test might have its uses in the evaluation of road oils and asphalts, it is not by any means the cure-all or universal specific which some of its proponents claim. Mr. Oliensis himself advances the test as merely a comparative test for heterogeneity and that is what it is.

It has been assumed that when an asphalt shows a negative reaction to the spot test that the sample is homogeneous. Strictly speaking this cannot be so since recent developments in research work on asphalt have shown that no bituminous material is homogeneous but is a colloid system consisting of a dispersion of asphaltenes in resins and oily constituents.

The arbitrary standards of heterogeneity which the Oliensis test creates might, as was mentioned before, have its place in the evaluation of bituminous materials, however in this article we wish to deal only with the detection of cracked oil.

All cracked oil will give a positive reaction to the spot test, but all oils which give a positive reaction to the test are not cracked. In the examination of a group of about 45 crude oils from various sections of the country we found 16 which were positive in varying degrees to the Oliensis spot test.

The test which we wish to advance as a test for cracked or burned bituminous material is the Resistance to Oxidation test as published in the Bulletin of the Kansas City Testing Laboratories*. The test was devised originally for use in the roofing trade but we have found, after 9 years of use and experimentation, that it is almost perfect as a bar to cracked oils. The test as used in New Mexico consists of coating about 4 inches of a 2 inch by 6 inch piece of thin sheet metal with the asphalt or distillation residue of the oil to be tested, and placing in an upright position in an oven. The strip is kept in the oven at a temperature of 275° F. for 15 minutes so that it drains and forms a uniform film on the metal. It is then removed and the temperature of the oven raised to 450° F. The strip is then placed back in the oven and allowed to bake at that temperature for one hour. At the end of this time the heat is turned off, the oven door opened, and the metal strip allowed to slowly cool in the oven to room temperature.

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*Bulletin No. 25. Page 693.

At the end of the test the surface or film on the metal strip should meet the following four requirements:

1. It should be brilliant and lustrous.
2. It should not be scaly and fragile.
3. It should adhere firmly to the metal.
4. It should not be dull and cheesy in texture.

Most cracked oils fall down on the requirements No. 2 and 3 which concern the tenacity of the film.

In all the years of our use of this test we have never found a cracked oil which would meet all the requirements listed, and we have been able to detect as little as 10% of cracked oil in a mixture with uncracked.

Uncracked oils will pass the test with one exception and that is oils or asphalts which contain large amounts of waxy or paraffinaceous bodies. Such bodies have been proven to be as deleterious to an asphalt as cracking.

In conclusion let me say that it has not been the intention in this paper to bring out a cure for all poor bituminous materials. We know that many uncracked oils give poor service in the road, and believe that the secret of getting a good quality asphalt or road oil lies in the selection of the crude oil from which it is to be made.

The intention of this article has been merely to set forth the results of some long range experiments which show clearly the poor quality of cracked bituminous material for road use and to advance a test which we believe should be in all road oil specifications as a comparatively quick, simple test which would be a bar to such cracked material.

Changes in Physical Characteristics of Paving Asphalt Cements and Their Relation to Service Behavior

By
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and Operations**

and
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In 1935 the Michigan State Highway Department adopted a specification controlling the penetration and ductility of bituminous materials after mixing and placing in the pavement. The Abson recovery method was specified for determining the characteristics of the recovered asphalt. During the four years since that time, the pavements built under these specifications have shown no indications of deterioration which cannot be definitely attributed to base failures. However, the research program maintained by the Department has been continued and these pavements have been surveyed each year, investigating their condition and removing samples for laboratory analysis. By such a program, an accurate inventory is kept by the Department indicating the condition of the existing pavement and enabling a study of mixture design for future bituminous construction.

In this paper the pavement survey made on pavements laid between 1934 and 1938 will be discussed. This will cover changes in the physical characteristics of the asphalt cement during mixing and placing operations and also annual changes due to weathering of the pavement. Consideration

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*The research described in this paper was under the supervision of the late Mr. J. D. Brown, Engineer of Bituminous Research, who contributed materially to the correlation and interpretation of the results obtained.

TABLE 1

OBSERVED CHANGES IN PENETRATION AND DUCTILITY OF THE BITUMEN IN VARIOUS BITUMINOUS CONCRETE PAVING PROJECTS

PROJECT			Characteristics of Asphalt Cement Recovered from Bituminous Mixture by Abson Method														
	Tests on Asphalt Cement Used		Immediately After Construction			1 Year After Construction			2 Years After Construction			3 Years After Construction			4 Years After Construction		
			Penetration			Penetration			Penetration			Penetration			Penetration		
Designation	Penetration of Bitumen*	Ductility, cm.	Reading	Decrease %**	Ductility, cm.	Reading	Decrease %**	Ductility, cm.	Reading	Decrease %**	Ductility, cm.	Reading	Decrease %**	Ductility, cm.	Reading	Decrease %**	Ductility, cm.
1	52	100+				32	38.5	(37)	21.5	58.6	6	21	59.6	5	18.5	64.4	5
2	53	100+				38	28.3	(57)	22	58.5	9	22	58.5	21	19.5	63.2	12
3	60	89							26.5	55.8	9	22	63.3	7	17	71.7	6
4	68	(90)	59	13.2	100+	41.5	39.0	150+	42.5	37.5	150+	36	47.1	109			
5	64	(53)	51.5	19.5	100+	39	39.1	115	34.5	46.1	88	31.5	50.8	51			
6	58	100+	45.5	21.6	(82)	24	58.6	21	24.5	57.7	16	22	62.1	19			
7	57	100+	49	14.0	100+	34	40.4	52	32	43.9	31	28	50.9	26			
Avg.	59		51	17		35	41		29	51		26	56		18	66	

* In the case of blends of each parts of petroleum and native asphalts containing mineral matter, the penetration has been calculated as the arithmetic average of the penetration of the petroleum asphalt and the mineral-free bitumen of the native asphalt.

** Per cent decrease in penetration is based on the penetration of the mineral-free bitumen in the original asphalt cement.

( ) Ductility values in these cases have not been corrected for the mineral matter in the bitumen.

will be given to the individual asphalts as reflected by the change in physical characteristics both in the pavement and in laboratory tests.

BITUMINOUS CONCRETE PAVEMENTS

Seven of the bituminous concrete paving projects constructed during 1934 and 1935 were selected for pavement survey, the results of which are given in Table I. The aging of the pavement is reflected by the decrease in penetration and ductility of the asphalt recovered from the pavement by the Abson recovery test. The Abson recovery test was not run on the pavement laid in 1934, since the test was not specified until June 5, 1935. Both straight petroleum asphalts and blends of petroleum asphalt and native asphalt are represented in this survey. Where blends of equal proportions of native asphalt and petroleum asphalt have been used, the penetration of the asphalt cement upon which the percentage decrease is based has been calculated as the arithmetic average of the penetration of the petroleum asphalt and the mineral-free bitumen of the native asphalt. Exact mixing temperature data is not available for the projects listed in Table I. The allowable limits specified were 275° to 375° F. between which, dependent upon various factors pertaining to the mixture characteristics and climatic conditions, the engineer designated the mixing temperature.

In general analysis of Table I, it is seen that the average decrease in penetration during construction is approximately 20 per cent, and 20 per cent, 10 per cent, and 5 per cent successively in the following three years. Data for the fourth year are available on too few projects to arrive at a reliable average figure.

In 1936 an experimental bituminous concrete paving project was conducted by the department. This project is of interest in the study of the physical characteristics of asphalt cements in that five petroleum asphalts and four blends of equal proportions of different petroleum asphalts with one native asphalt were used on nine different sections of this project. This gives a comparison of the individual physical characteristics of the various asphalts.

The data of the survey of this experimental project are tabulated in Table 2. Test values are shown for the recovered asphalt immediately after construction and after one and two years aging. It will be noted that again the average decrease in penetration of the asphalt cement during mixing and laying is approximately 20 per cent but ranges from 2 to 47 per cent for the individual asphalts. Also the average decrease in penetration for the first and second years after laying is approximately 20 and 10 per cent. The asphalt cement in this pavement at the end of two years has, therefore, approximately 50 per cent of its original penetration. This actual decrease in penetration and per cent decreases in penetration is shown graphically in Figures 1 and 2 respectively.

Asphalts Nos. 10 and 12 deserve special recognition because of the individually different aging characteristics they present. Asphalt No. 12 has a much greater loss in penetration during construction than the average. This material is a processed asphalt composed of a blend of

TABLE 2

OBSERVED CHANGES IN PENETRATION AND DUCTILITY OF THE BITUMEN IN BITUMINOUS CONCRETE PAVEMENT

Original Asphalt Cements			Characteristics of Asphalt Cement Recovered from the Bituminous Mixture by Abson Method									
				Immediately After Construction			1 Year After Construction			2 Years After Construction		
				Penetration			Penetration			Penetration		
Designation	Penetration of Bitumen[1]	Ductility cm.[3]	Mixing Temp. ° F.	Reading	Decrease %[2]	Ductility cm.[3]	Reading	Decrease %[2]	Ductility cm.	Reading	Decrease %[2]	Ductility cm.
8	64	100+	320	55	14.1	150+	37.5	41.4	134+	33.5	47.7	56
9	68	100+	320	50	26.5	150+	42.5	37.5	150+	31.5	53.7	150+
10	53	100+	325	52	1.9	147	47.0	11.3	150+	42.0	20.8	128
11	67	150+	350	52	22.4	101	36.5	45.5	62	35.0	47.8	47
12	72	77	340	38.0	47.2	150+	35.0	51.4	84+	31.0	56.9	52
13	68.5	100+	320	50.0	27.0	115	40.5	40.9	150+	29.5	56.9	90
14	68.5	100+	325	49.0	28.5	150+	47.0	31.4	150+	34.5	49.6	150+
15	62.5	100+	330	43.5	30.4	60+	39.0	37.6	150+	38.5	38.4	135
16	70.0	75	330	51.0	27.1	150+	38.0	45.7	130	35.5	49.3	99
Avg.	66.0			49.0	25.0		40.0	38.0		35.0	47.0	

[1] In the case of blends of equal parts of petroleum and native asphalts containing mineral matter, the penetration has been calculated as the arithmetic average of the penetrations of the petroleum asphalt and mineral-free bitumen of the native asphalt.

[2] Per cent decrease in penetration is based on the penetration of the mineral-free bitumen in the original asphalt cement.

[3] Ductility values not corrected for the mineral matter in the bitumen.

a very hard bitumen and a relatively fluid flux which is subject to a higher evaporation loss at plant mixing temperatures. This accounts for this high initial decrease in penetration. The penetration drop of this fluxed asphalt is compensated for, at least in part, by its higher original

Aging of Bituminous Concrete Pavement as
Reflected by Drop in Penetration of Recovered Asphalt

Figure No. 1

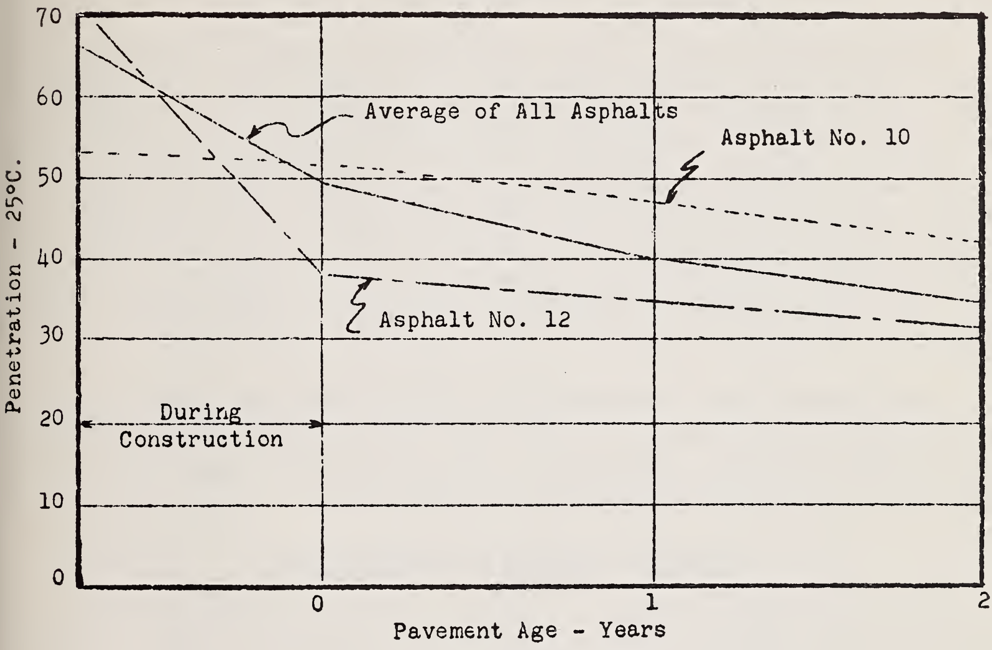
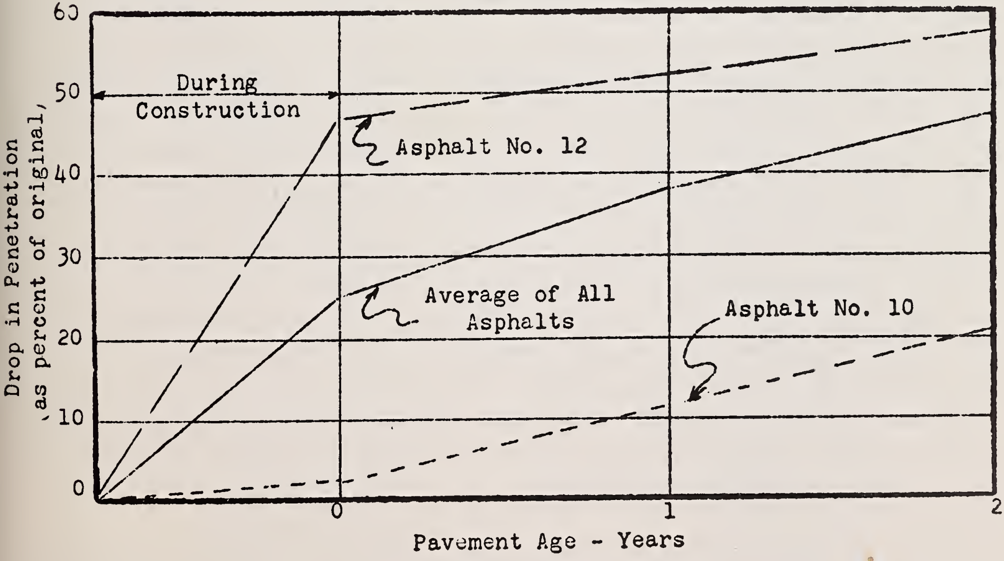


Figure No. 2



penetration. It will also be noted in Figure 2 that the rate of hardening of Asphalt No. 12 in the two-year period following construction as shown by the slope of the lines is of a lesser degree of magnitude than the average of all asphalts used on this project. If this trend continues, the penetration of Asphalt No. 12 will be approximately the same as the average penetration in another year or so.

The behavior of Asphalt No. 10 with respect to penetration drop has been so unusual that it has been plotted separately on Figures 1 and 2.

The penetration drop during construction and after the first year's aging was so much below average that the results were first eliminated from consideration as probable errors in testing or from sample contamination. With the compilation of the 1938 survey, however, it was shown that the rate of hardening over the two-year period has been so uniform that there can be little doubt of reasonable accuracy. The conclusion reached is that these two asphalts differ from the others used on this project. The fact that asphalts vary in susceptibility to oxidation has been previously described in a research report of this department.*

The limited period of observation and the uniformly good condition of the pavement on the project at this time precludes any forecast of serviceability. Further investigation will be carried out until such a time that some definite conclusions may be drawn.

SHEET ASPHALT PAVEMENTS

Data on a limited number of sheet asphalt pavements are shown in Table 3. These data cover pavements laid in the years 1934 to 1937. The decrease in penetration of blended native and petroleum asphalts when used in the pavement is computed on the basis of the mineral-free bitumen in the blended asphalt. It will be noted that the annual decreases in penetration and ductility are not nearly as uniform as with bituminous concrete. Such results as are available do indicate that while the decrease in penetration of the bitumen in sheet asphalt and bituminous concrete is approximately the same during construction the decrease in penetration in four years for the sheet asphalt is less than the first year's drop in penetration of the bitumen in the bituminous concrete pavements. This hypothetically may be due to several causes. The first explanation given is the difference in the character of the bituminous mix. Another explanation offered is that these sheet asphalt pavements sampled are located in large cities or their environs where there is a large volume of relatively slow-moving traffic which may deposit in its course an oil film which fluxes with the bitumen and causes it to retain its original consistency and resist aging. Since the data at present are rather limited, further annual surveys which will be continued on these projects may give a more conclusive explanation.

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* A Study of Asphalt Recovery Tests and Their Value as a Criterion of Service Behavior, V. B. Steinbaugh, Deputy Commissioner-Chief Engineer; J. D. Brown, Engineer of Bituminous Research, Michigan State Highway Department, Proceedings, Association of Asphalt Paving Technologists, December 6-8, 1937, p. 138.

TABLE 3

OBSERVED CHANGES IN THE PENETRATION AND DUCTILITY OF THE BITUMEN IN VARIOUS SHEET ASPHALT PAVING PROJECTS

PROJECT			Characteristics of Asphalt Cement Recovered from Bituminous Mixture by Abson Method.														
	Tests on Asphalt Cement Used		Immediately After Construction			1 Year After Construction			2 Years After Construction			3 Years After Construction			4 Years After Construction		
			Penetration			Penetration			Penetration			Penetration			Penetration		
Designation	Penetration of Bitumen*	Ductility cm.	Reading	Decrease %**	Ductility cm.	Reading	Decrease %**	Ductility cm.	Reading	Decrease %**	Ductility cm.	Reading	Decrease %**	Ductility cm.	Reading	Decrease %**	Ductility cm.
17	63	100+							34	46.0	99+	39.5	37.3	148+	30.5	51.6	75
18	55	100+							35	36.4	97	38	30.9	93	41	25.5	110
19	63	(60)	50	20.7	(100+)	43	31.8	131+	43	31.8	139+	45	28.6	113+			
20	63	(93)	47.5	24.6	150+				30	52.4	58						
21	66	150+	50	24.2	150+				52	21.2	150+						
22	62	(111)	52.5	15.3	150+				50	19.4	150+						
23	62	(84)	48	22.6	150+	42.5	31.5	90									
Avg.	62		50	21		43	32		41	35		41	32		36	38	

* In the case of blends of equal parts of petroleum and native asphalt, containing mineral matter, the penetration has been calculated as the arithmetic average of the penetrations of the petroleum asphalt and the mineral-free bitumen of the native asphalt.

** Per cent decrease in penetration is based on the penetration of the mineral-free bitumen in the original asphalt cement.

( ) Ductility values in these cases have not been corrected for the mineral matter in the bitumen.

CORRELATION OF MIXTURE CHARACTERISTICS AND SERVICE BEHAVIOR

Of all the bituminous pavements laid since 1934, only Projects Nos. 1 and 2 listed in Table 1 have shown serious deterioration. Some slight cracking was noticeable within two years after construction on these projects and it has progressed rapidly to the present time, during the fifth year, when raveling of the surface has become quite pronounced.

It will be noted in Table 1 that the characteristics of the recovered bitumen alone are not sufficient to explain differences in service behavior, since the recovered asphalt from Project No. 3 is very similar to that from Projects Nos. 1 and 2. Since Project No. 3 is in relatively good condition as compared with Projects Nos. 1 and 2, other factors must enter into the picture. Two factors shown in the average mechanical analysis of various bituminous concrete pavements as shown in Table 4 may explain the difference in service behavior. Approximately 63 per cent of the bituminous mixture used on Projects Nos. 1 and 2 was retained on the No. 10 sieve while on Project No. 3 only 54 per cent of the bituminous mixture was retained on the same sieve. Past experience has indicated that when much over 50 per cent, possibly up to 55 per cent stone, is present in such a mixture the bitumen-sand-mineral filler mortar is insufficient to provide either impermeability to air and moisture or sufficient mechanical strength to resist stresses, causing cracking particularly after several years of aging has increased the brittleness of the asphalt cement. Until recent years the more or less general practice of applying seal coats over stone mixes of this type undoubtedly contributed greatly toward protecting the pavement beneath from rapid oxidation of the asphalt and reduced the tendency toward raveling by filling the surface voids with bitumen. The elimination of the seal coat to provide a uniform non-skid surface makes it doubly important that the maximum size and amount of stone be not too great. In 1936, the reduction of the Department's specification for maximum size of stone for bituminous concrete from ¾ inch to ½ inch as noted in Table 4 for Projects Nos. 24 and 25 seems to have eliminated the segregation which promotes

TABLE 4
AVERAGE ANALYSIS OF BITUMINOUS CONCRETE PAVEMENTS

	MIXTURE ANALYSIS								
	Fraction Retained, Per Cent								
Project Designation	¾″	½″	¼″	No. 10	No. 40	No.80	No. 200	% Passing No. 200	Bitumen %
1 & 2	1	22	34	6	5	12	7	8	5.3
3	0	13	37	4	6	22	6	6	5.8
6	0	16	34	5	6	22	6	6	5.7
5	0	11	38	6	6	19	7	7	5.7
4	0	10	39	8	7	17	8	6	5.6
7	0	9	38	7	7	13	13	7	5.6
8 thru 16	0	17	32	3	6	25	5	7	5.6
24	0	2	46	5	8	21	6	7	5.5
25	0	2	48	7	8	17	6	7	5.6

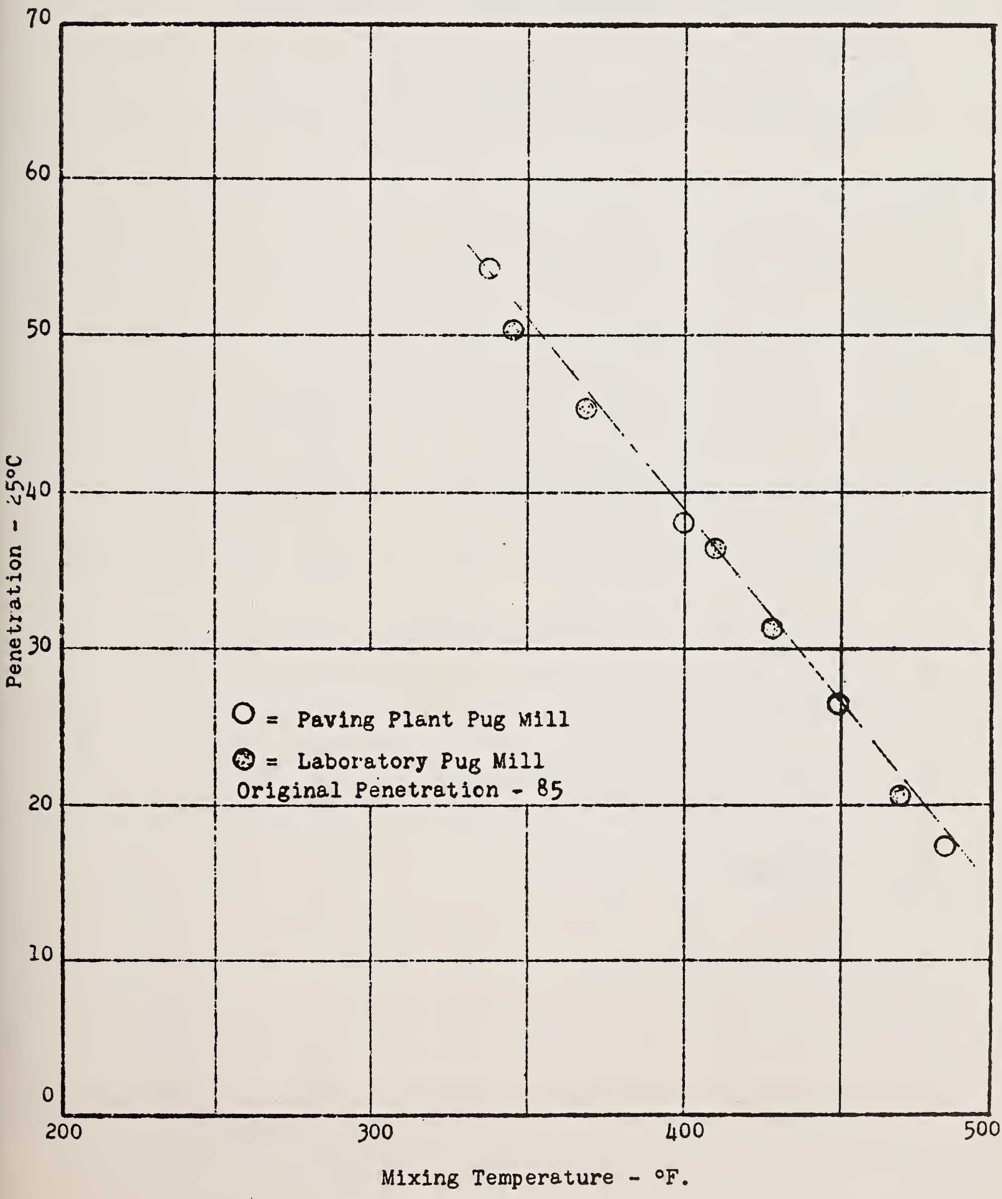
raveling and should aid in impermeability. Sufficient time has not elapsed to judge precisely the comparative serviceability.

As to the bitumen content of the mix itself, the one half or one per cent additional bitumen in Project No. 3 over that of Projects Nos. 1 and 2 should account for much of its greater serviceability. All types of bituminous mixtures should contain the maximum amount of bitumen consistent with stability in place. For the type now being discussed this should be probably not less than 6 per cent. Proper mixture design plays an important part in the serviceability of bituminous pavements.

In discussing mixture characteristics it is well to note at this point that a number of investigators have found that when the penetration of

Figure No. 3

Relation of Mixing Temperature of Bituminous Concrete to Penetration of Recovered Asphalt



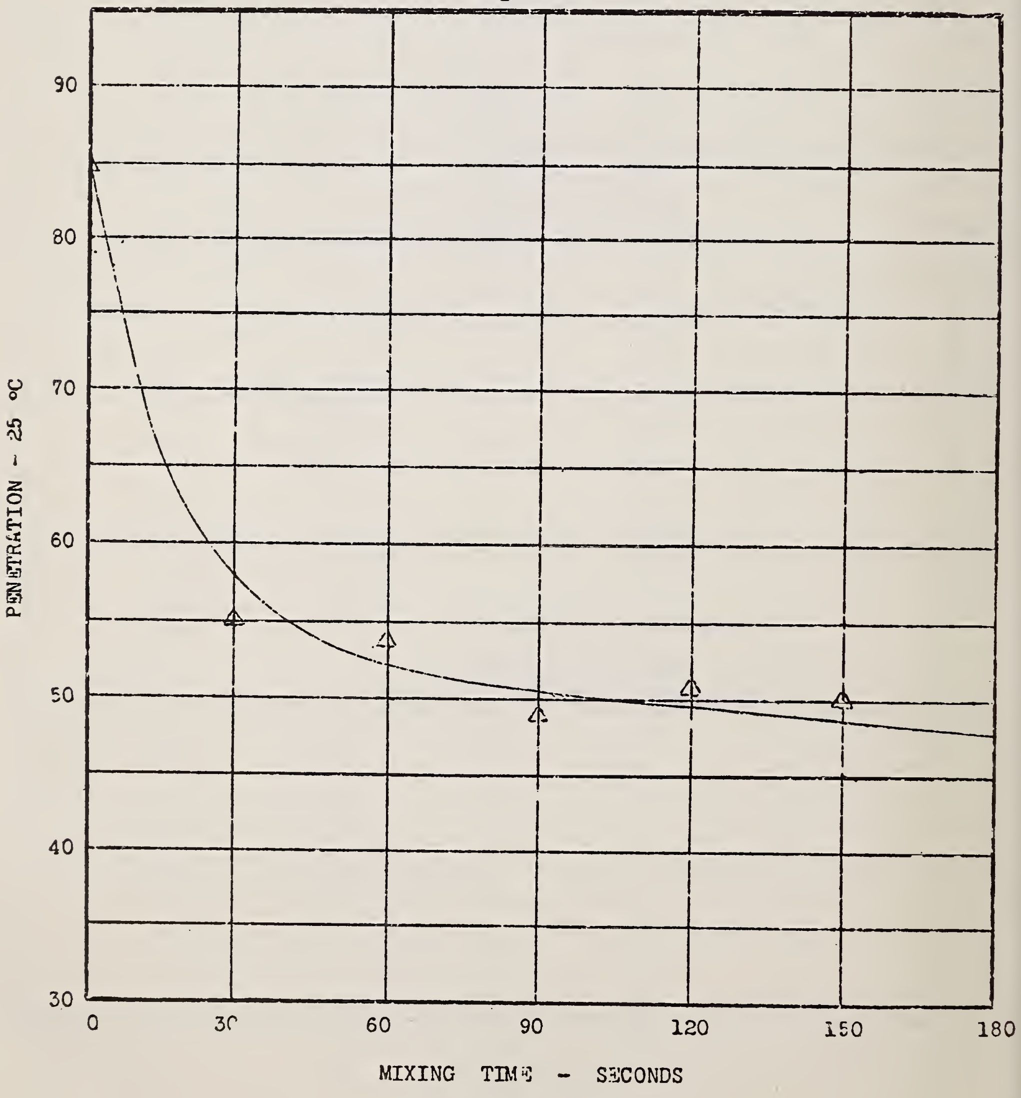
the recovered bitumen falls much below 30, trouble with cracking may be expected to develop. If this is the case, several of the bituminous concrete pavements listed in this survey may soon show sign of distress. Further survey of these pavements will verify this contention. Little mention has been made of the drop in ductility values determined in this survey because of the wide variation. Too little data are available to determine what the safe lower limit of ductility is to indicate serviceability.

The use of softer asphalts in bituminous concrete which was put into effect in 1936 should be a decided advantage in adding life to the pavements. At that time the specified limits of penetration of the asphalt cement were increased from 50-60 to 65-75. There is one point which has

PENETRATION vs MIXING TIME

Bituminous Concrete - Plant Mixed

Figure No. 4



probably not been fully considered in the use of softer asphalt cements. This is the greater hardening obtained at the same mixing temperatures. The use of an 85 penetration asphalt on a bituminous concrete mixture for resurfacing showed an average drop in penetration of 30 to give a penetration of 55 for the bitumen in the pavement after mixing and placing. The percentage reduction in penetration was approximately 35 per cent as against approximately 20 per cent decrease for bituminous concrete mixed of lower penetration asphalt at appreciably higher temperatures.

LABORATORY AND FIELD TESTS ON HARDENING OF ASPHALT CEMENTS DURING MIXING

The importance of mixing temperatures was investigated in both the laboratory and field using the 85 penetration asphalt previously mentioned. Identical mixes were made up in both a paving plant and laboratory pug mill to determine the effect of temperature on the penetration of the bitumen recovered from the mix. The results are shown in Figure 3. Duplication of results obtained from paving plant mixes by those from the laboratory pug mill furnishes excellent proof of the necessity for accurate and intelligent temperature control of the mixing process, especially when minimum penetration and ductility requirements are specified for the bitumen extracted from the mix.

For comparison of the importance of mixing time with mixing temperature another series of tests were made in the field paving plant using the same mix and asphalt but varying the time of wet mixing. The temperature of the mix was controlled at 340° F. plus or minus 10° F. The results of this test series are shown in Figure 4. In examining this curve it will be seen that the greatest drop in penetration occurs in the first 30 seconds. In the next 150 seconds the penetration dropped only 10 points as compared with approximately 27 points in the first 30 seconds of mixing. The decrease in penetration in the first 30 seconds is evidently caused primarily by volatilization of the lighter components of the soft bitumen. The importance of mixing temperature over mixing time is shown in these field and laboratory tests.

CONCLUSIONS

This paper has reported on the changes in asphalts occurring during mixing and laying and in the pavement over a period of several years after laying. The changes observed confirm the research done in this laboratory showing that different asphalts were hardened at different rates by oxidation in a laboratory test procedure as described in a previous report*. The relatively short period of time that many of the projects listed in the survey have been laid precludes any definite analysis of superiority for any particular type of asphalt.

The two cases of definite signs of failure in a bituminous concrete pavement occurred in pavements that showed excessive drops in both penetration and ductility but these failures could not be definitely related to the hardening of the asphalt alone. Two other projects with equal hardening showed no apparent failures at this time. An analysis of the mixture design of the various pavements in this survey showed various differences which may compensate to some extent for excessive hardening of the bitumen. Proper choice of maximum size and gradation of the aggregate and selection of the proper bitumen content, therefore, must be emphasized as factors to pavement life.

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*A Study of Asphalt Recovery Tests and Their Value as a Criterion of Service Behavior, V. B. Steinbaugh, Deputy Commissioner-Chief Engineer; J. D. Brown, Engineer of Bituminous Research, Michigan State Highway Department, Proceedings, Association of Asphalt Paving Technologists, December 6-8, 1937, p. 138.

It has been shown that while time and temperature of mixing are both factors in the hardening of asphalt the temperature factor is much more important and must be more carefully controlled especially when asphalt cements of higher penetrations are being used as is the present trend. With proper control of mixing temperatures and time, the use of softer asphalts will give a softer bitumen in the pavement after mixing and laying. This has the advantage in lengthening the aging period before the bitumen in the pavement has become hard and brittle and increases the life of the pavement before cracking will occur.

An Evaluation of Recent Trends Toward the Use of Softer Asphalts

By

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There recently has become evident a trend toward the use of softer asphalts in hot mix bituminous concrete pavements. It is believed that this trend is proper and justifiable, and that the use of softer asphalts will result in better and more durable pavements. It seems probable that with proper plant control the old specification limits of penetration of the asphalt cement of 51-60 in hot climates and 61-70 in cold climates may be profitably increased by at least 10 points, and possibly as high as 101-120 penetration if justified by future experience. This change toward a softer asphalt may well require renewed attention to other specifition details in order to obtain the full advantages of the softer asphalt.

The arguments for the use of softer asphalts have recently been cognitively stated by Buchanan* as follows:

"Current studies seem to indicate that softer asphalt cement (higher penetration) is desirable in hot-laid, plantmix pavements to attain better performance, longer life and less cracking not only in cold climates but also in hot climates where susceptibility to hardening has been rather acute. Whereas it was formerly reasoned that low penetration asphalt cement was required for hot weather service to provide stability at high temperatures, research of the past few years has shown that aggregate is the major factor supplying stability to an asphaltic mixture. This conclusion is further supported by the high stability and relative crack-free performance of liquid asphaltic plant-mix and road-mix pavements in hot climates."

"Present thought trends toward asphaltic concrete using 101-120 penetration asphalt rather than 51-60 penetration which has been almost standard for hot climates. As further information develops from experience, asphalts even softer than 101-120 penetration may be preferred. Many hot-laid plant-mix pavements are now being constructed using 151-200 penetration asphalt or SC-6 liquid asphaltic material.'' Baldock* describes an open type asphaltic concrete, using a macadam aggregate mixture and an RC-4 liquid asphalt instead of paving asphalt, which experience in Oregon demonstrates to be easily placed, durable and to retain its non-skid characteristics. ''Performance records of these will furnish valuable information bearing on the question of how far the trend toward softer asphalt should go. At present, however, it seems entirely conservative to select 60 penetration as the minimum to be used in asphaltic concrete (in warm climates). As softer asphalts are used, mixing temperatures may be lowered, which apparently will mean less hardening of asphalt during construction and probably will provide a greater margin against premature aging and brittleness.''

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*Asphalt Forum, Vol. II, No. 5.

These opinions are supported by the following conclusions of Hubbard and Gollomb.†

''1. Extreme hardness of asphalt in an asphalt pavement is conducive to cracking.

2. Contact with air is a principal cause of hardening of asphalt.

3. All asphaltic products harden progressively and with appreciable rapidity when continuously exposed in thin films to the action of air.

4. Rapidity of hardening is a function of film thickness, temperature and time,—the thinner the film, the higher the temperature and the longer the period of contact with free air, the greater will be the degree of hardening.

5. Thin films of all liquid asphaltic road materials, as well as asphalt cements may be expected to harden to low penetration brittle asphalts within a few years' time if not adequately protected from contact with free air.

6. At hot-mix paving plants asphalts may be expected to harden appreciably during their combination with mineral aggregate under customary plant operation.

7. From data at present available it appears that the following expectancy of cracking due to hardness of the asphalt cement present in the pavement, exists in climates where freezing temperatures are common through the winter.

   (a) Asphalt less than 20 penetration—bad cracking.
   (b) Asphalt higher than 30 penetration—no cracking.
   (c) Asphalt 20 to 30 penetration—uncertain.

8. An asphalt of 50 penetration, irrespective of source, may very likely be reduced to as low as 35 penetration during preparation of a hot-mix thus bringing it dangerously near the limit

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*Asphalt Forum, Vol. III, No. 1.

†Proceedings of Association of Asphalt Paving Technologists, December 1937, page 182.

of penetration at which the pavement may be expected to develop some cracks.

9. In hot-mix construction the softer the original asphalt the less is it likely to harden dangerously during the mixing operation. For such construction an asphalt of 60 penetration or softer is strongly indicated as a means of preventing hardening to a dangerous extent during the mixing process.
10. Progressive hardening of asphalt films in a dense thoroughly compacted paving mixture is exceedingly slow. If, however, the mixture is readily permeable to air, dangerous hardening may occur within a relatively short period.
11. To insure long life, prevent cracking and possible disintegration in an asphalt pavement or wearing course, due to hardness of the asphalt, the following five rules should be strictly observed:
    - (1) Use as soft an asphalt as possible without reducing stability below the minimum required to prevent displacement under traffic.
    - (2) Use as high a percentage of asphalt as possible, without reducing stability below the minimum required to prevent displacement under traffic. In this way film thickness is increased to the maximum practical extent and air permeability decreased.
    - (3) At paving plants prepare mixtures at the lowest practicable temperatures.
    - (4) Compress all asphalt paving mixtures thoroughly so that they will be as impermeable to air as possible.
    - (5) Seal coat all compressed mixtures which are readily permeable to air, and thus prevent free circulation of air within the mixture."

These conclusions of Hubbard and Gollomb, and of Buchanan, are supported by the experience in Oregon, previously referred to. The Oregon experience demonstrated that the grading of the aggregate is an important matter in the construction of an asphaltic concrete using a cut-back asphalt (RC-4) instead of asphalt cement. Their current specification follows:

Passing	Retained On	Per Cent
½″ screen	¼″ sieve	40-50
¼″ sieve	10 mesh sieve	30-40
10 mesh sieve	200 mesh sieve	8-20
200 mesh sieve		0-2
Cutback asphalt (RC-4)		4-6

Williamson* in discussing trends in asphalt construction states that "the trend during the past year in the United States has been definitely toward the use of softer asphalts. Engineers are becoming conscious of the fact that the consistency of asphalt binders as represented by penetration tests have relatively little relationship to stability of pavements in which they are used. Factors of design other than the penetration of asphalt, principally ratio of asphalt and aggregate, govern more importantly in developing necessary stability. In the construction of airport surfaces we find this trend even more pronounced, naturally because of the little amount of traffic, and the need for resistance to weathering.''

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*Roads and Streets, January 1939, page 35.

As a summary of the opinions of a large number of paving engineers throughout the United States Macatee* ''shows that there is almost complete unanimity of thought that asphalt cements of somewhat higher penetration (60-70 or more) are desirable,'' and that this opinion is also shared by asphalt technologists.

The case for the use of softer asphalts thus seems to be established. The chief justification for further consideration of this topic at this conference lies in the opportunity afforded by this conference to disseminate these opinions and bring them to the attention of paving engineers who have not previously been informed, and who, therefore, continue to conform to the older established practice.

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*American Road Builders Association, 1938, page 501.

Interpretation of Current Tests on Bituminous Materials in the Light of Recent Research

By

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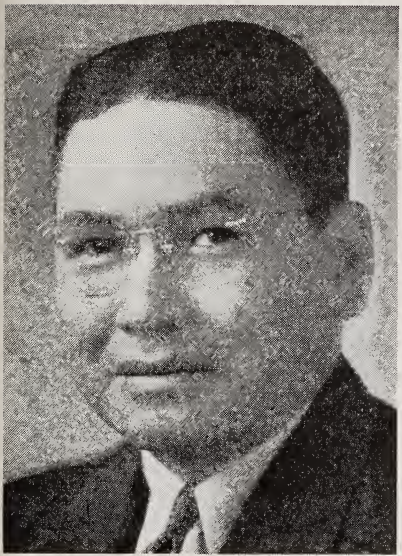

INTRODUCTION

The purpose of this report is to present a discussion of correlations made between composition, expressed in specific terms, and the results obtained with tests now used on asphalts of the slow curing type. Although the report is limited to this type of materials, it will be noted that some of the findings may also apply to other types of asphalts.

No summary or discussion of the reports of other investigators is presented because those interested in the subject are no doubt familiar with reports such as those of Lewis & Hillman[1] and others. Furthermore, this report concerns itself with the presentation of data which does not appear elsewhere and which is presented with the intent of inviting attention to the possibility of placing interpretations other than those which are ordinarily placed on the results of current tests.

In order that this report may not be unduly long, details involved in the work are left for presentation in some other manner.

SCOPE OF THE INVESTIGATION

The data herein presented were obtained in the course of conducting an investigation jointly with Professor O. A. Hougen of the Department of Chemical Engineering, University of Wisconsin.

The original scope of the investigation may be summarized as follows:

1. To find an explanation for the differences in service behavior of slow curing asphalts meeting the same specifications.
2. To examine critically specifications for slow curing asphalts in order to determine the value of the various requirements; to ascertain their relation to behavior and durability in service.

3. To determine the influence of chemical and physical properties of slow curing asphalts upon their adhesion and resistance to stripping by water from aggregates.
4. To study the effect of composition on the properties of slow curing asphalts.

The complete results of the investigation are published in a bulletin of the University of Wisconsin College of Engineering.

Samples Tested

One of the most serious handicaps confronting investigators not connected with the industry is the obtainment of samples of systematically varying composition. Lacking samples of this nature, it would be necessary to take innumerable samples of commercially available materials. Then, values for a range of compositions could be obtained by interpolating the results of testing such samples. A procedure of this kind would be burdensome and subject to a number of fundamental errors. To overcome the difficulty, large samples of the following five different SC asphalts from different sources, produced by different methods of refining were selected.

1. An uncracked, steam-refined Texas residue, naphthenic.
2. An uncracked, steam-refined Mid-Continent residue, paraffinic.
3. A slightly cracked residue, aromatic.
4. An uncracked Mexican residue, naphthenic.
5. A highly cracked residue, aromatic.

By means of solvents, the samples were separated into fractions identified as asphaltenes, resins, and oily constituents. These fractions were systematically recombined into synthetic samples which were then tested for various properties. The separation of the fractions was made in a large-scale laboratory plant designed by A. J. Hoiberg formerly of the University of Wisconsin.

The separation was based on the fractionation of the products according to molecular weight by means of a series of solvents of different internal pressures. Hexane was used to precipitate a high molecular weight fraction, the asphaltenes, and liquid propane was employed for separating resins from a lower molecular weight fraction, the oily constituents. These fractions do not represent definite hydrocarbons but are rather arbitrary separations defined by the method of separation and by the solvent. The fractions represent three distinct ranges of molecular weight, about 2000 for the asphaltenes, about 1000 for the resins and about 500 for the oily constituents. By employing a large number of solvents more fractions could be obtained, but in order to simplify interpretations and in order to establish relatively simple correlations with properties, the separation was limited to the three fractions mentioned.

A number of precautions were taken while making the synthetic samples in order to minimize or avoid possible changes in the fractions. Original samples and synthetic samples of similar composition were then compared by means of tests. All properties, with the exception of viscosity at low temperatures, were found to be alike. The discrepancies in viscosities at low temperature are attributed to the age factor; original samples were tested several months after their manufacture, while the synthetic samples were tested shortly after they were made.

Original and synthetic samples were tested by means of standard methods of test. Correlations between composition and the various properties tested were made by means of triangular charts which permit the simultaneous presentation of the percentages of the three fractions used to obtain given compositions.

Specific Gravity of Blended Samples

In general it was found that the specific gravity is a simple additive property which can be represented, for the samples studied, by a general equation of the form

$$s = xa + yr + zo$$

where

s = specific gravity of the blended sample
x = specific gravity of the asphaltene fraction
y = specific gravity of the resin fraction
z = specific gravity of the oily constituent fraction
a = percentage of asphaltenes
r = percentage of resins
o = percentage of oily constituents

The cracked residues showed the highest specific gravities, which was in agreement with the aromatic nature and low hydrogen content of the materials; the products from the Mexican and Texas crudes showed a striking similarity, and the products from the Mid-Continent crude showed the lowest specific gravities, which agreed with the paraffinic nature and high hydrogen content of the products.

In attempting to establish correlations with other properties, the most striking correlation was found with resistance to stripping of the bitumen from aggregates. Increases in resistance to stripping with increases in specific gravity were found to occur regardless of the bitumen or the aggregate used. The increase was definite up to about a value of 1.000 for specific gravity. Beyond this value the resistance either increased slightly or remained constant. In making the tests it was noted that the resistance to stripping of samples containing no asphaltenes was consistently lower than the resistance of samples containing asphaltenes.

The results obtained in making approximately 5000 tests indicate that specific gravity is an important factor in evaluating resistance to stripping.

Viscosity of Blended Samples

Viscosities were measured at three temperatures—104°, 122° and 194° F.—by means of the rise in capillary tube method. The results were studied not only as expressions of viscosity, but also as expressions in the asphalt viscosity index of Traxler[2], in the viscosity-temperature susceptibility index of Nevin and Krchma[3], and in a viscosity-temperature slope index.

It was found that viscosity is influenced greatly by the asphaltene content and to a lesser extent by the resin content. Generally, the viscosity was about doubled by a 5% increase in asphaltenes with constant resin content and by a 10% increase in resins with constant asphaltene content. The cracked residues showed low viscosity, with changes in viscosity following closely changes in composition. With similar composition, the products of the Texas and Mexican crudes showed much higher viscosities. The products of the Mid-Continent crude showed the least agreement between composition and viscosity, possibly because of the paraffinic nature of the products.

None of the three indices mentioned were found to be adequate in identifying the source of the crude from which the samples were prepared. In general, it appears that composition affects the values obtained for the various indices much more readily than origin and quite often the values varied erratically in the series of samples prepared from the same material.

Viscosity susceptibilities are greatly influenced by the asphaltene content.

In correlating viscosity values with specific gravity and resistance to stripping characteristics it appears that the viscosities now in current use are not sufficiently high.

Percentage of Residue of 100 Penetration

The residue obtained by means of the procedure described in the A. S. T. M. method for the determination of the percentage of residue of specified penetration has been assumed to represent the residue as it may exist after the road mat has undergone a period of exposure. This assumption has not been accepted in general and such specifications as those of the Asphalt Institute[4] do not recognize the test. Since it is desirable to examine residues of liquid asphalts, distillation tests have been proposed to replace the aforementioned test. Distillation tests, however, have not worked satisfactorily for the type of slow curing asphalts which is used in Wisconsin and for this reason it has been necessary to either use the method of A. S. T. M. serial designation D243-36 or to find a procedure other than distillation which would make available residues of given consistency.

Obviously the first step in studying the matter was a consideration, as complete as possible, of the method for determining percentage of residue of specified penetration. In this work, the usefulness of the method was examined by field and laboratory studies. The field studies were made by means of periodic examinations of the bitumen extracted from a mat located on a specially chosen road in the State Trunk System. The extracted bitumen was examined for changes in viscosity and for changes in composition expressed by percentages of asphaltenes, resins and oily constituents. After making these examinations for a period of sixty weeks of service, the following conclusions have been reached:

1. Evaporation of the oily constituents present in the original asphalt chiefly accounted for the changes in consistency occurring in the bitumen in the mat.
2. At the end of the sixty weeks, the consistency of the bitumen in the mat approached that corresponding to a penetration of 150.
3. After evaporation took place, the gain in consistency was found to be gradual and it was calculated that an appreciable period of time would elapse before a consistency of 100 penetration would be reached.
4. The changes that take place in the course of evaporating a sample of SC asphalt to a predetermined consistency in the laboratory are similar to changes that take place in the same SC asphalt while in a road surface; both are produced chiefly by evaporation. Therefore, the residue obtained by means of the method for the determination of the percentage of residue of specified penetration is of value because it makes possible the performance of tests which may serve as an evaluation of the bitumen found in a road mat after a period of service.

By testing, it was determined that the ratio of resins to asphaltenes did not change materially in the course of reducing the asphalt to consistency of 100 penetration or during the period of sixty weeks of weathering, therefore, the changes that take place in either case are believed to be due to evaporation inasmuch as if the oily constituents had changed to resins or the resins to asphaltenes the changes could not have taken place without at sometime affecting the ratio of asphaltenes to resins. No doubt changes due to processes other than evaporation take place but such changes are not as outstanding as those caused by evaporation.

As a result of the laboratory and field studies, it is believed that the residue obtained in the method for determining percentage of residue of specified penetration furnishes the means for examining residues corresponding to residues left in the road mat after a period of exposure. It should be noted that the residue considered in this discussion is that obtained by means of the apparatus specified in A. S. T. M. serial designation D243-28T and not the residue obtained using the apparatus specified in D243-36. The first specifies a hemispherical bowl, much larger in size of exposed surface area than the six ounce gill can specified by the latter method. Using the bowl, the length of time required to complete the evaporation is much shorter than that required by the gill can.

Ductility of Residue of 100 Penetration

The laboratory of the State Highway Commission of Wisconsin has tested literally thousands of samples for ductility at 77° F. No correlation has been found between the results of the tests and behavior in service. Further, this property does not appear to correlate with other properties of the asphalts. The property, therefore, appears to have no significance but if it has, such significance is not clear.

Ductilities at 39.2° F. 5 cm/min. were made on the synthetic samples considered in this study. The highly cracked residues showed a ductility of 5.0 with a composition of about 19% asphaltenes, 1% resins and 80% oily constituents. Three or four other compositions also showed ductility but the values were only 1.0, and in one case 4.0. The partly cracked residues showed, through a range of compositions, low temperature ductility varying between 3.0 and 4.0 The products of the Mexican and Texas crudes were much alike in low temperature ductility: through a range of compositions, samples from these crudes showed ductility of 10.0. The products from the Mid-Continent crude also showed ductility through a range of compositions; the ductility, however, was not as high at any composition as that shown by the Mexican or Texas crude products.

Desirable limits for ductility at low temperature and the method for making the test are mooted questions. Possibly the only way to settle them would be through service tests covering a range of asphalts of varying ductilities.

Solubility in Naphtha

The synthetic samples were tested by means of hexane and naphtha (86-88° Baume, at least 85% distilling between 35 and 65° C.) A comparison of the results indicated that the solubility in naphtha was slightly greater than that obtained with the hexane; also it was found that the insoluble in naphtha increased somewhat with increases in resin content. It is believed, therefore, that the naphtha precipitates small fractions of resins along with the asphaltenes. For similar composition, the percentage of naphtha insoluble was the same for products obtained from the different crudes.

Fluidity Factor

Originally this test was intended to identify materials of naphthenic nature as it is believed that products of this nature have, in general, higher durability. Otherwise, the test was not intended to be a means for identifying "good" from "bad" asphalts.

In these studies it was found that regardless of composition the blends in the series of cracked asphalts had fluidity factors of values lower than 130; a great many of them had negative values. The products from the Mid-Continent crude showed a large area of compositions in which the fluidity factor values ranged from 130 to 150. The products from the Mexican and Texas crudes showed fluidity factor values greater than 150 over all the range of composition excepting through a small area containing approximately 5% asphaltenes, 0 to about 60% oily constituents, and 40 to 85% resins.

Spot Test

Unquestionably the Oliensis spot test has filled a need in asphalt technology. The results that are herein reported, however, indicate that it is possible to obtain positive spot tests on materials that have not been overheated, cracked, blown, or which do not have wax, sludge or incompatible fractions. In studying the products of the Texas and

Mexican crudes it was found that for the former, compositions of 5 to 45% asphaltenes, 20% resins and 60 to 100% oily constituents would show positive spot tests. Likewise, the products from the Mexican crude having a composition of 0 to 45% asphaltenes, 0 to 20% resins and 45 to 100% oily constituents showed positive spot tests. These compositions were homogeneous in the sense that neither in their original condition nor at any time during the performance of tests any of the samples showed separations or precipitation of fractions. The products from the Mid-Continent crude showed a definite area in which all compositions gave a positive spot test and within which a number of compositions showed non-homogeneity. The cracked residues, as expected, showed positive spot tests throughout the entire range of compositions.

Possibly the areas of positive spot tests for the Texas and Mexican crude products do not include compositions of commercially available products, therefore, the spot test would still be significant in its practical application.

Resistance to Stripping

Resistance to stripping of bitumen from aggregates was measured by means of a modified Nicholson[5] procedure and by the Riedel and Weber method. Three types of aggregates were considered: quartzite of acid nature, dolomitic limestone of basic nature, and glacial gravel containing an admixture of sedimentary, metamorphic and igneous aggregates.

On the basis of approximately 5000 tests, the following conclusions have been reached:

(a) In general it was found that the results given by the Riedel and Weber method were in agreement with the results of the water-stripping test although in correlating resistance to stripping with specific gravity, the results of the former method were erratic.

(b) The complicated properties of the sodium carbonate solution make the Riedel and Weber test more difficult to handle and more difficult to interpret than the water-stripping test.

(c) Asphalts of paraffinic nature were found to adhere better to the acid aggregate while asphalts of naphthenic nature adhered better to the basic aggregate. The cracked residues showed particularly good adhesion to the acid aggregate.

(d) The differences in resistance to stripping towards aggregates of different nature disappeared when specific gravities of the bitumens exceeded values of 1.000.

(e) Evaluations of resistance to stripping from aggregates of one size or from aggregates containing a limited range in sizes do not correlate with evaluations of resistance to stripping from aggregates containing a range of sizes such as are found in road mats.

LABORATORY WEATHERING TESTS

Laboratory weathering cycles were planned to simulate weather conditions found in Wisconsin.

Thin films, of bitumen of uniform thickness, 0.7 mm., were applied to aluminum panels, exposed for four hours to ultra-violet lights from a mercury arc lamp, for nine hours to a spray of lake water, for two hours to freezing temperatures and then were allowed to dry for nine hours at room temperature. The cycles were repeated for ten days and the specimens were examined at the end of the first day and at two day intervals thereafter for appearance, loss in weight and asphaltene content.

The results were disappointing because no correlation could be found between laboratory tests and behavior in service. It was found that neither appearance nor asphaltene content followed changes taking place in the road. Furthermore, in the laboratory tests the effect of ultra-violet light was found to be out of proportion to its effect on the bitumen in a road mat. It has been concluded, therefore, that laboratory weathering tests involving exposures to ultra-violet radiation, to freezing temperatures and to water sprays do not duplicate results obtained in service.

REFERENCES

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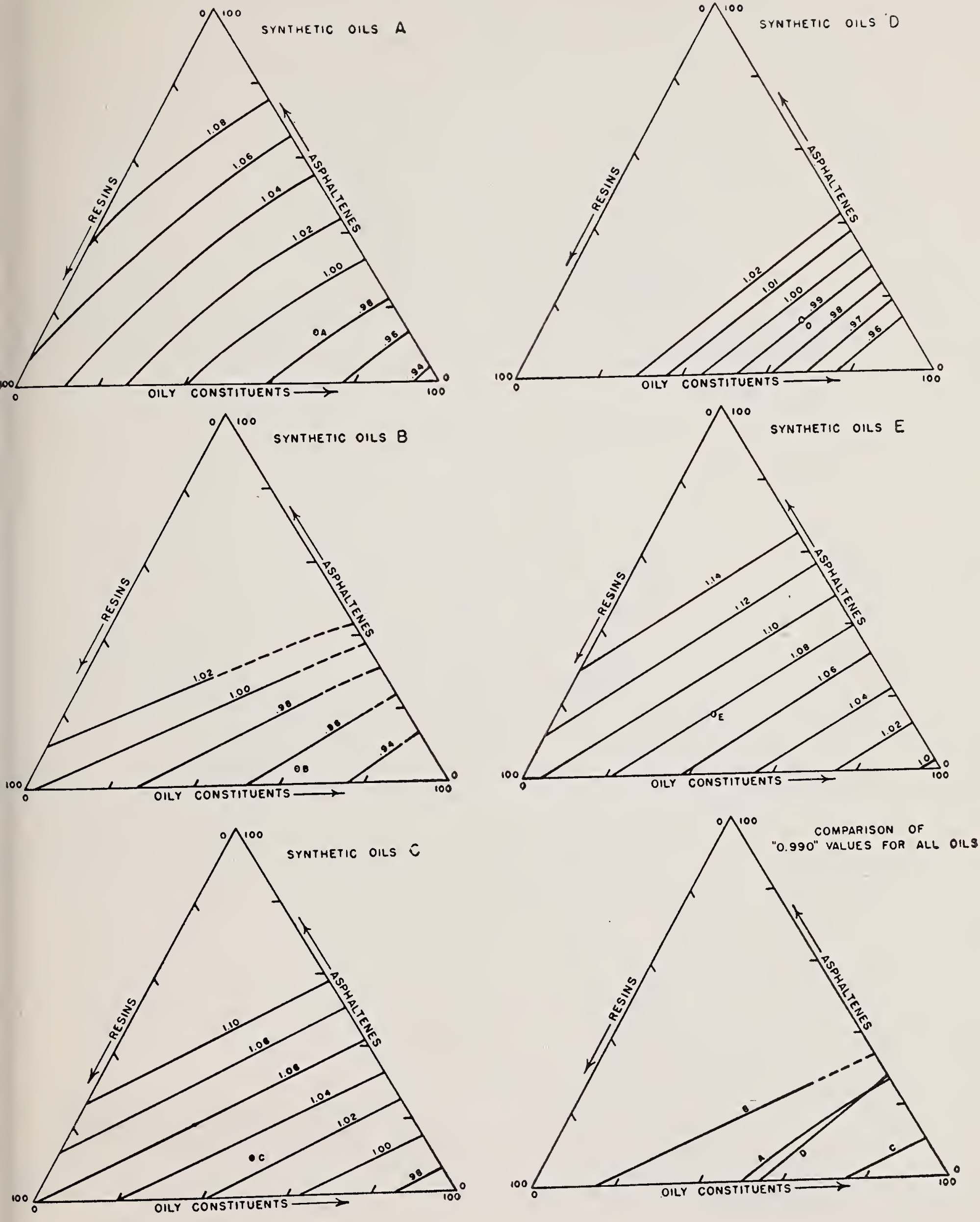
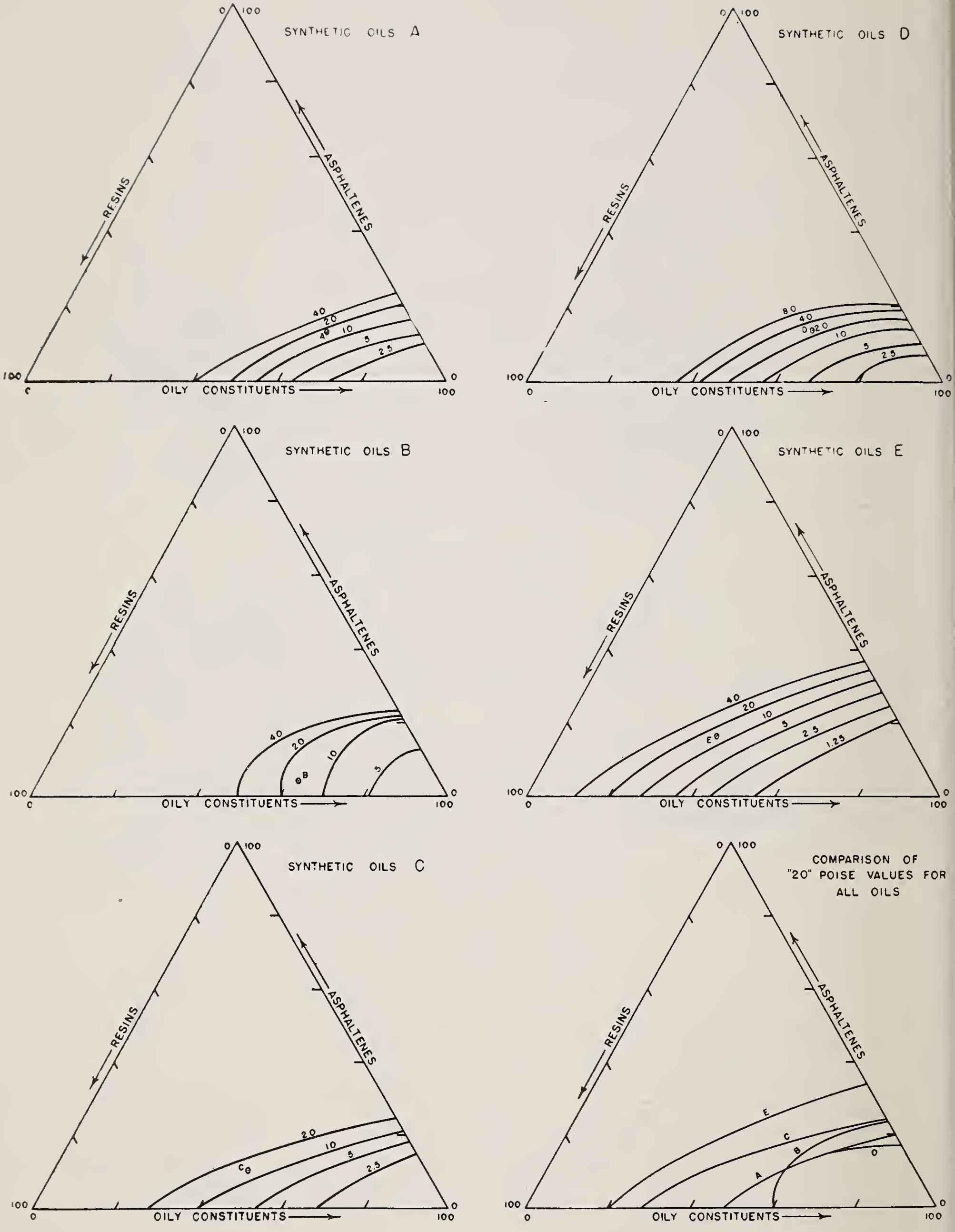
2. Schweyer, Coombs, and Traxler, Proc. Am. Soc. Testing Materials, 36, Part II.
3. Nevitt and Krchma, Ind. Eng. Chem. Anal. Ed., 9, 119 (1937).
4. Asphalt, Pocket Reference for Highway Engineers, Edition of 1937.
5. Nicholson, v. Assoc., Asphalt Paving Tech., Jan. 1932, 43.



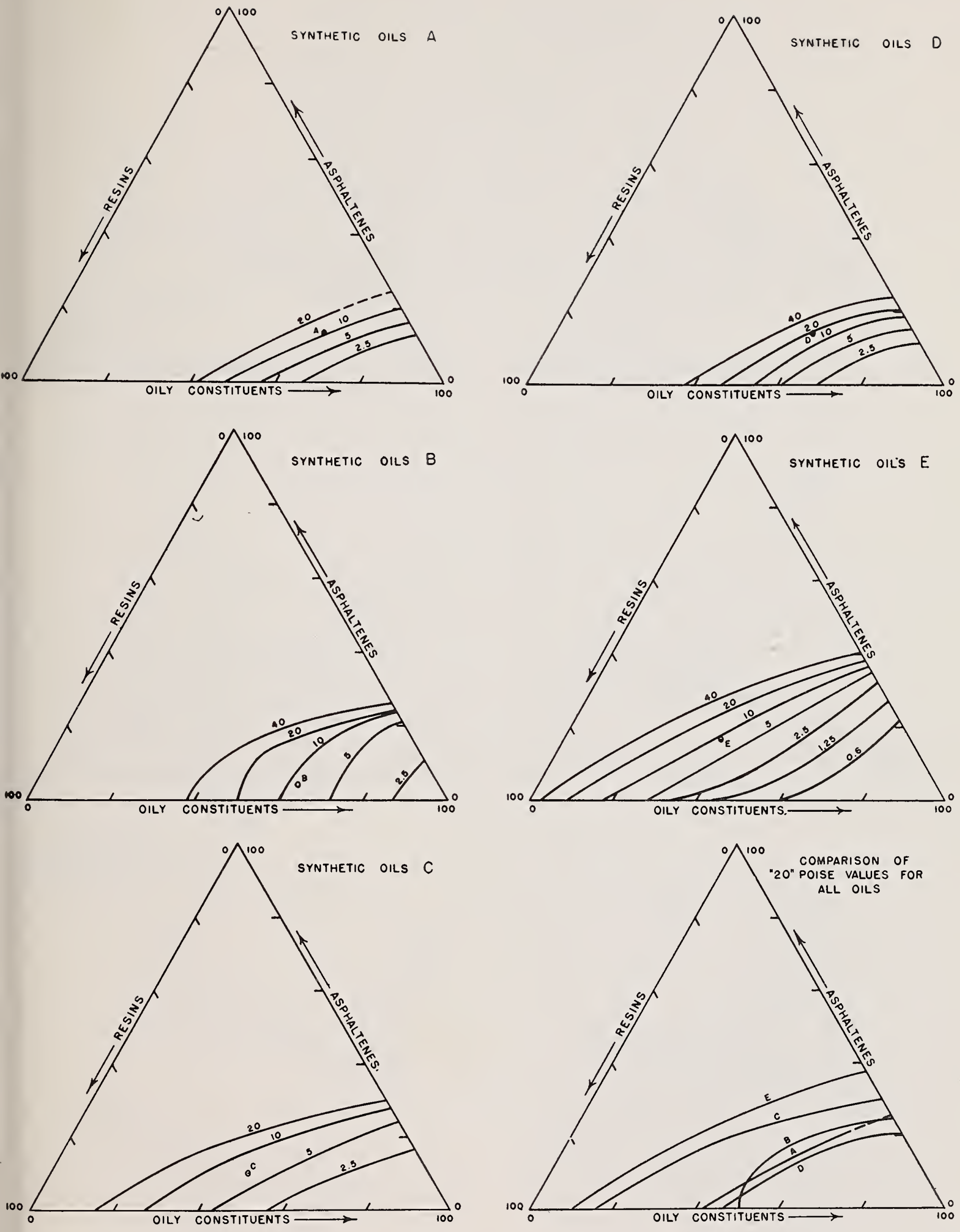
SPECIFIC GRAVITY AT $\frac{60°F}{60°F}$

FIGURE 3



VISCOSITY IN POISES AT 122° F

FIGURE 5



VISCOSITY IN POISES AT 140° F

FIGURE 6

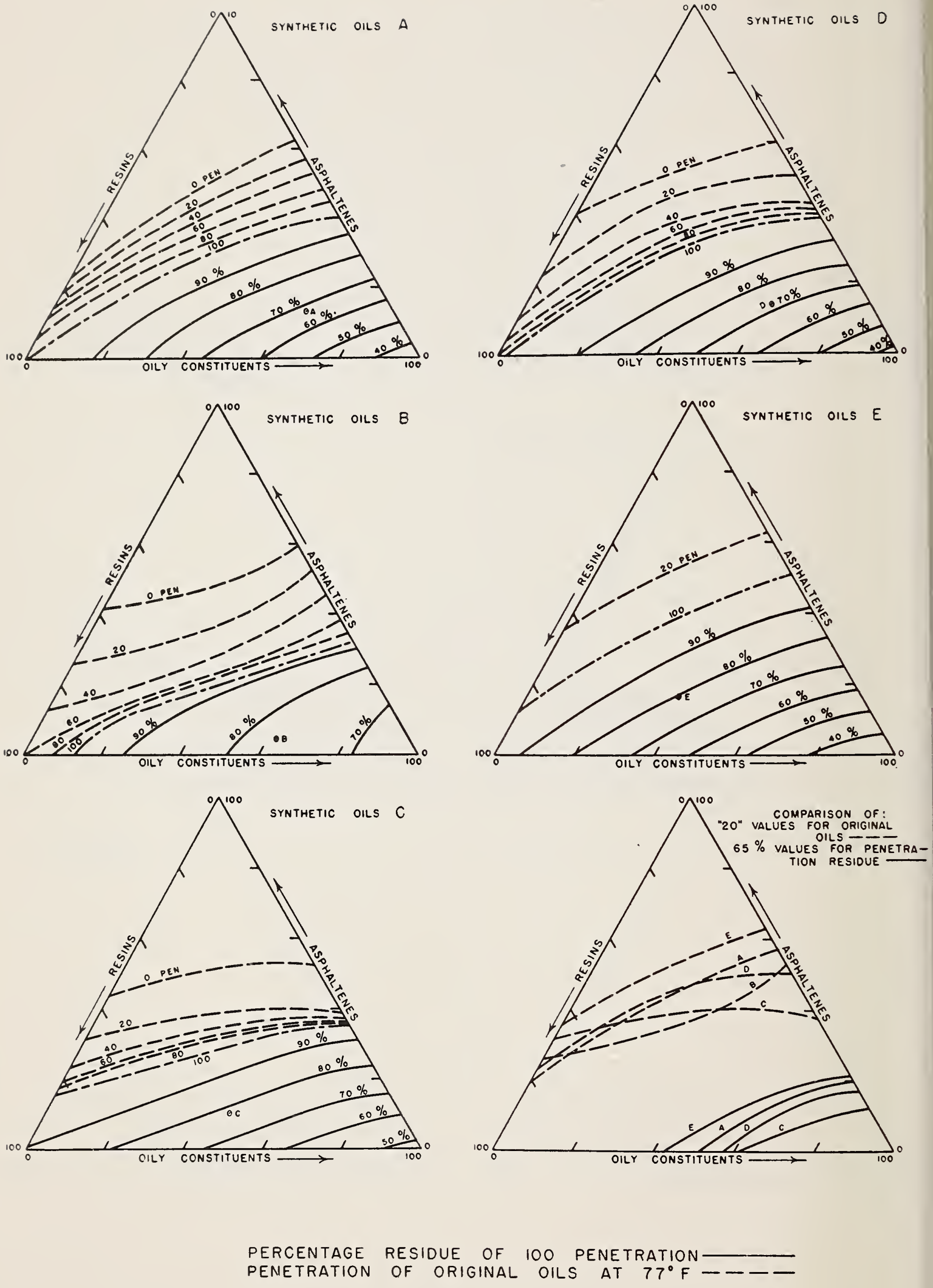
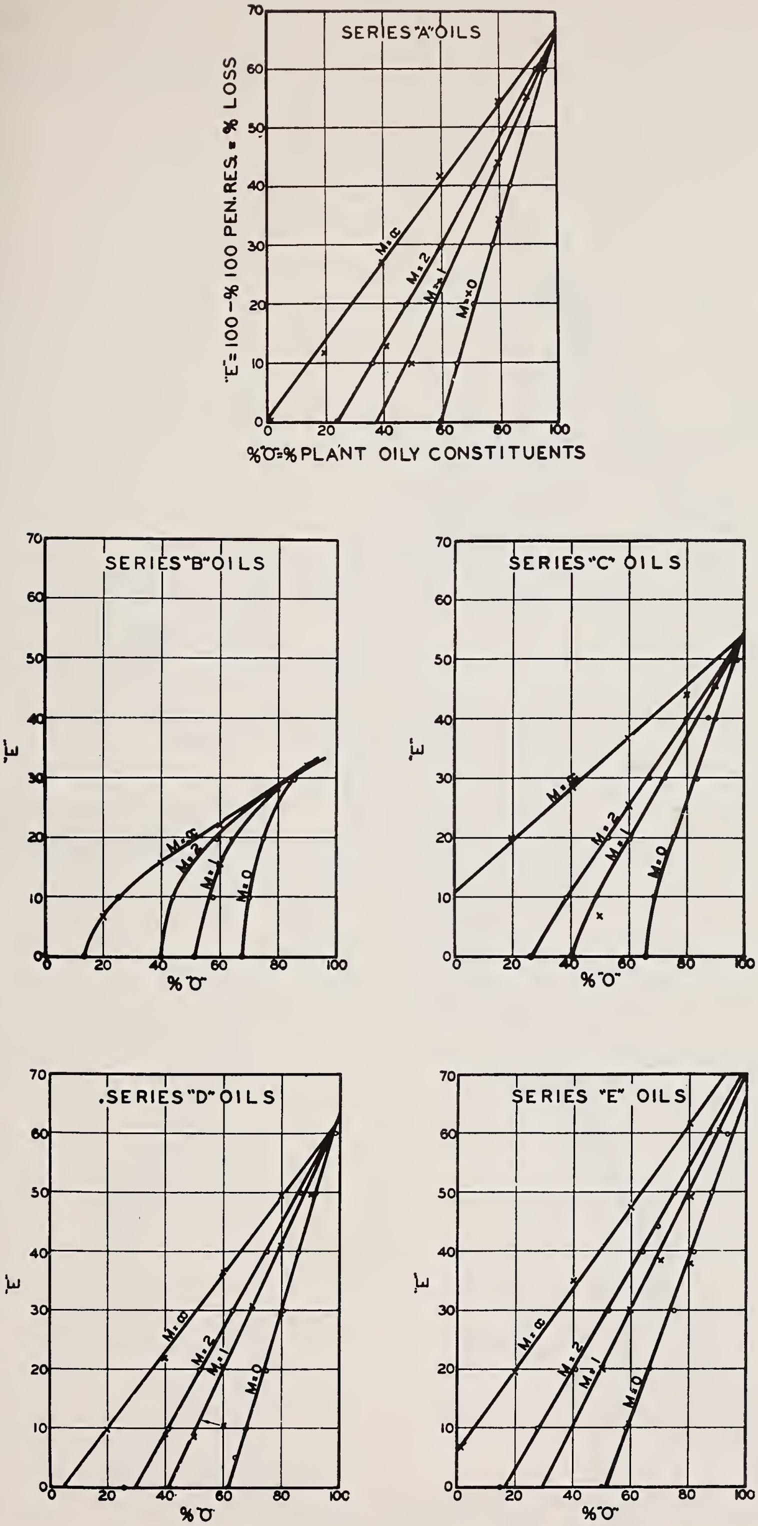

SYNTHETIC OILS A
SYNTHETIC OILS B
SYNTHETIC OILS C
SYNTHETIC OILS D
SYNTHETIC OILS E
RESINS
ASPHALTENES
OILY CONSTITUENTS
0 PEN
20 PEN
20
40
60
80
100
90 %
80 %
70 %
60 %
50 %
40 %
A
B
C
D
E
0
100
COMPARISON OF:
"20" VALUES FOR ORIGINAL OILS
65 % VALUES FOR PENETRATION RESIDUE
PERCENTAGE RESIDUE OF 100 PENETRATION
PENETRATION OF ORIGINAL OILS AT 77° F

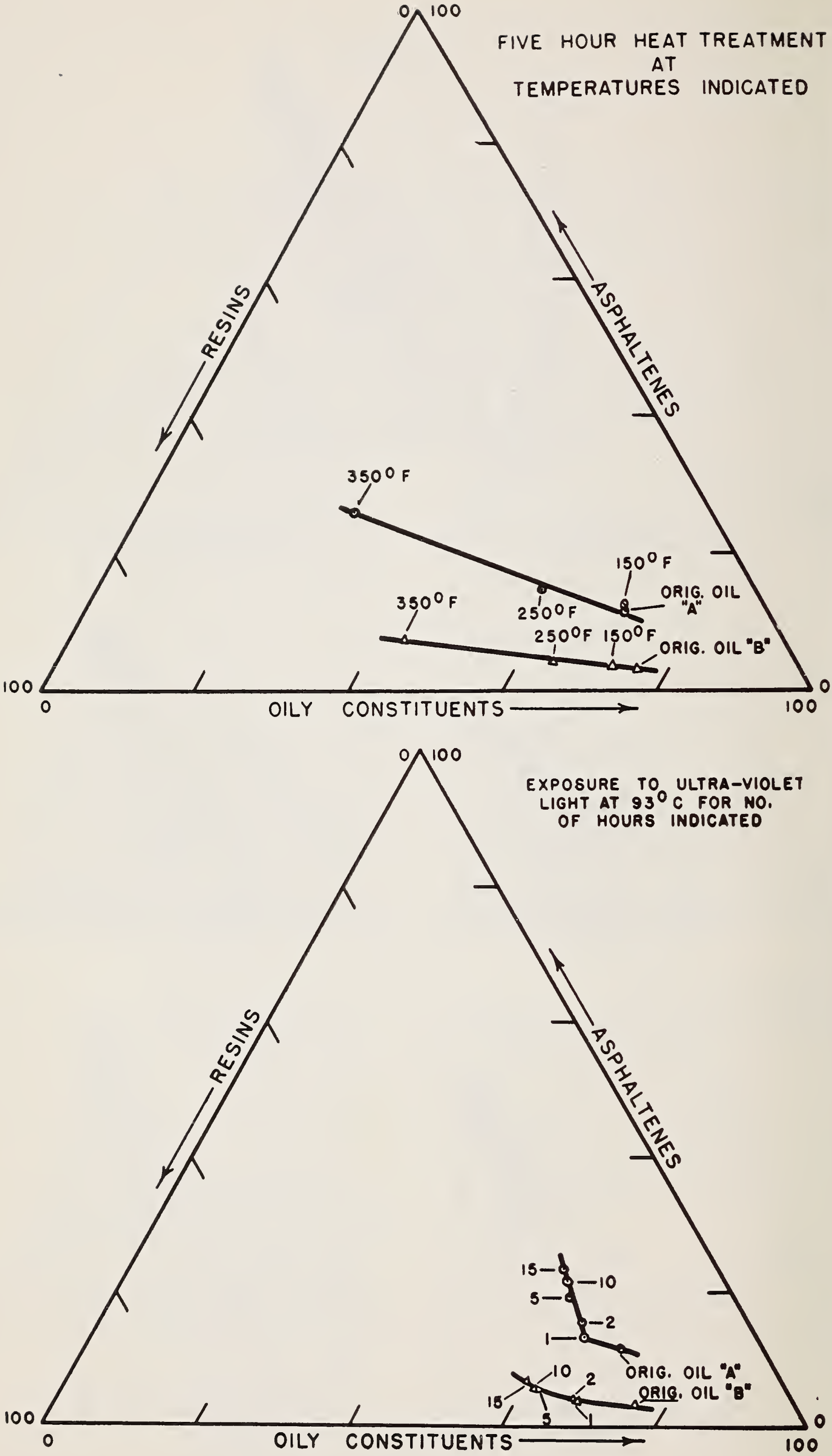
FIGURE 14



LOSS DURING 100 PENETRATION RESIDUE TEST (E) VS.
% PLANT OILY CONSTITUENTS (%"O") AT VARIOUS
RATIOS OF PLANT RESINS TO PLANT ASPHALTENES (M)

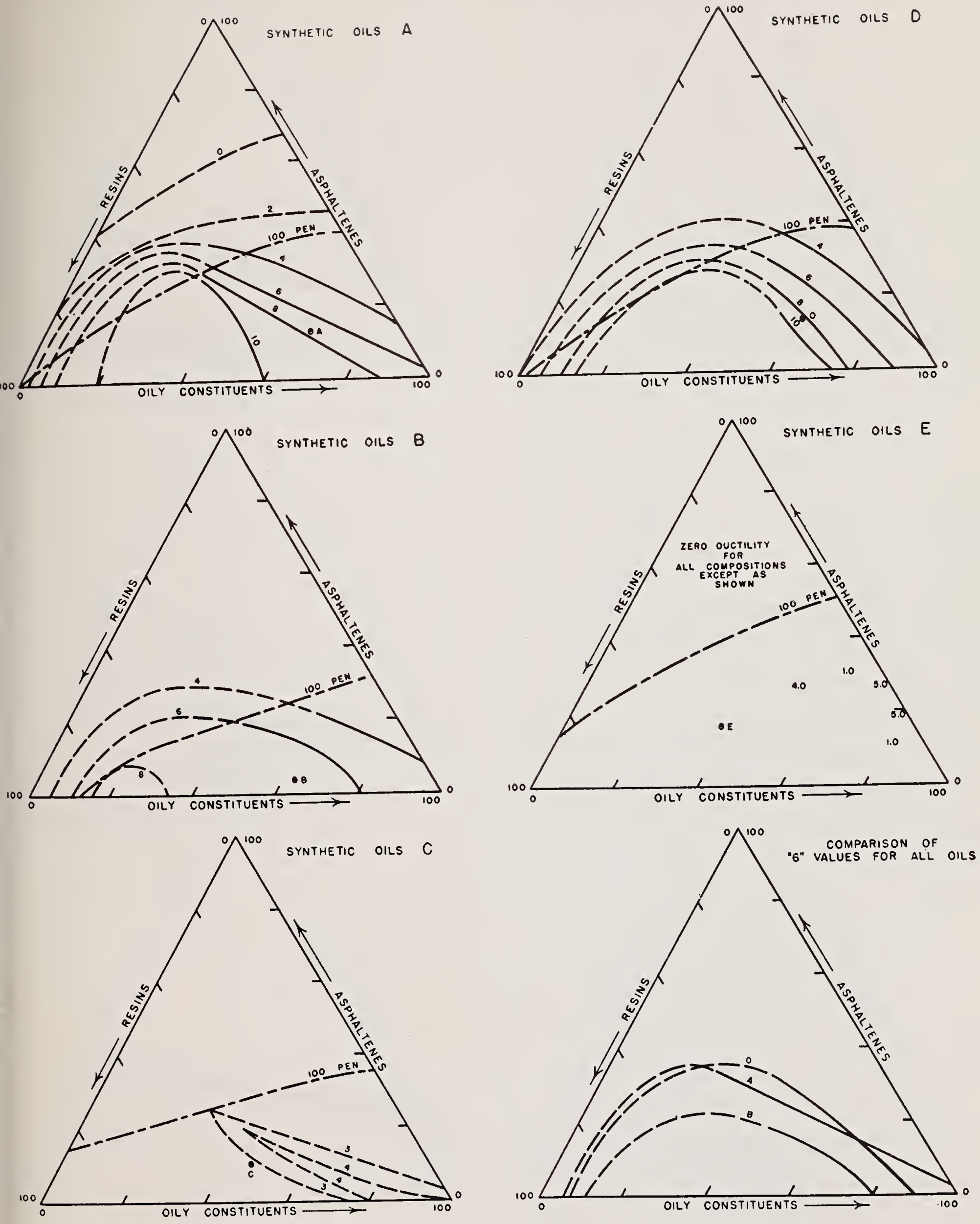
x = EXPERIMENTAL DATA
o = DATA FROM TRIANGULAR DIAGRAMS

FIGURE 14 A



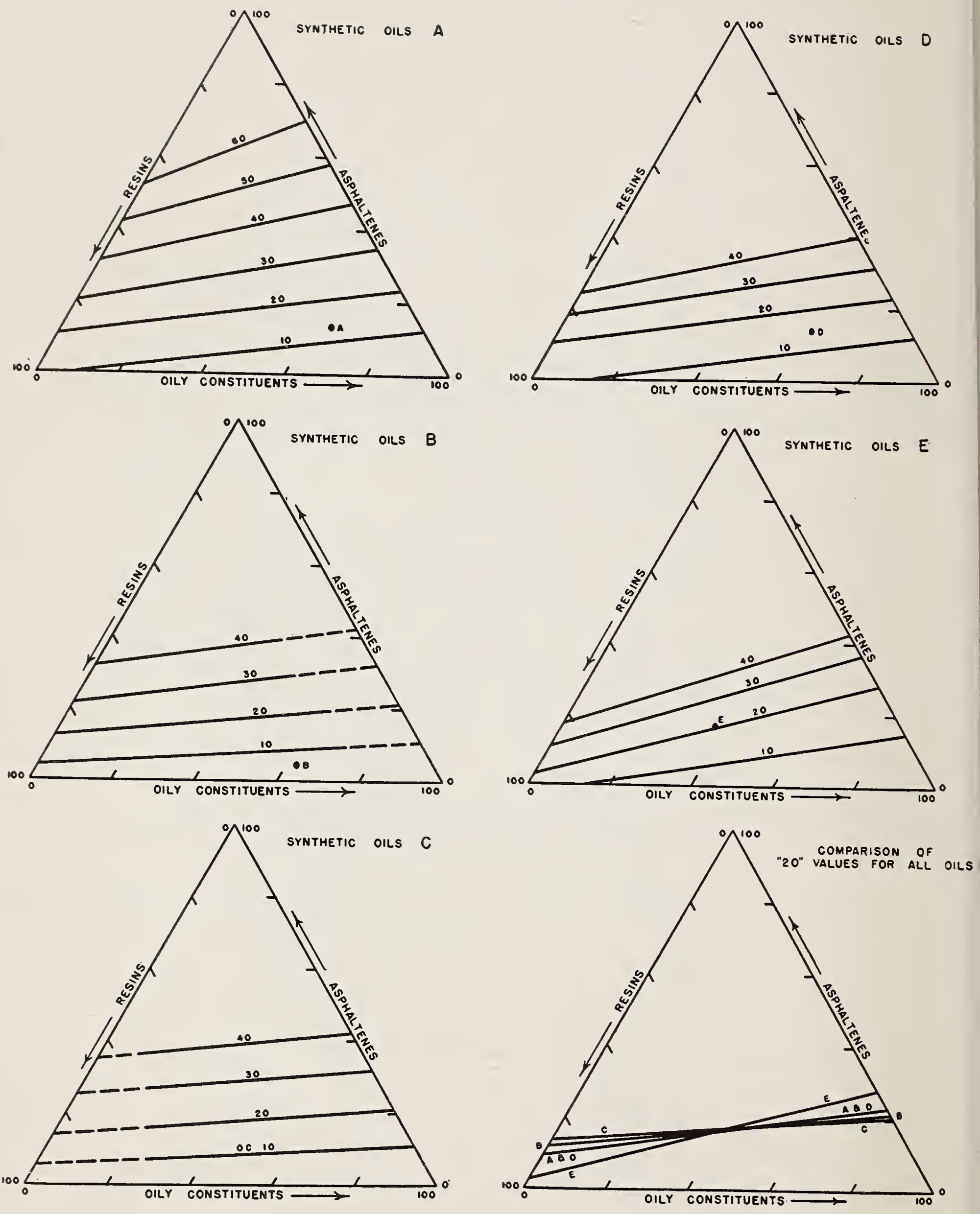
COMPOSITION (LAB. ANALYSIS) AFTER HEAT AND ULTRA-VIOLET LIGHT TREATMENTS OF OILS "A" AND "B"

FIGURE 14 B.



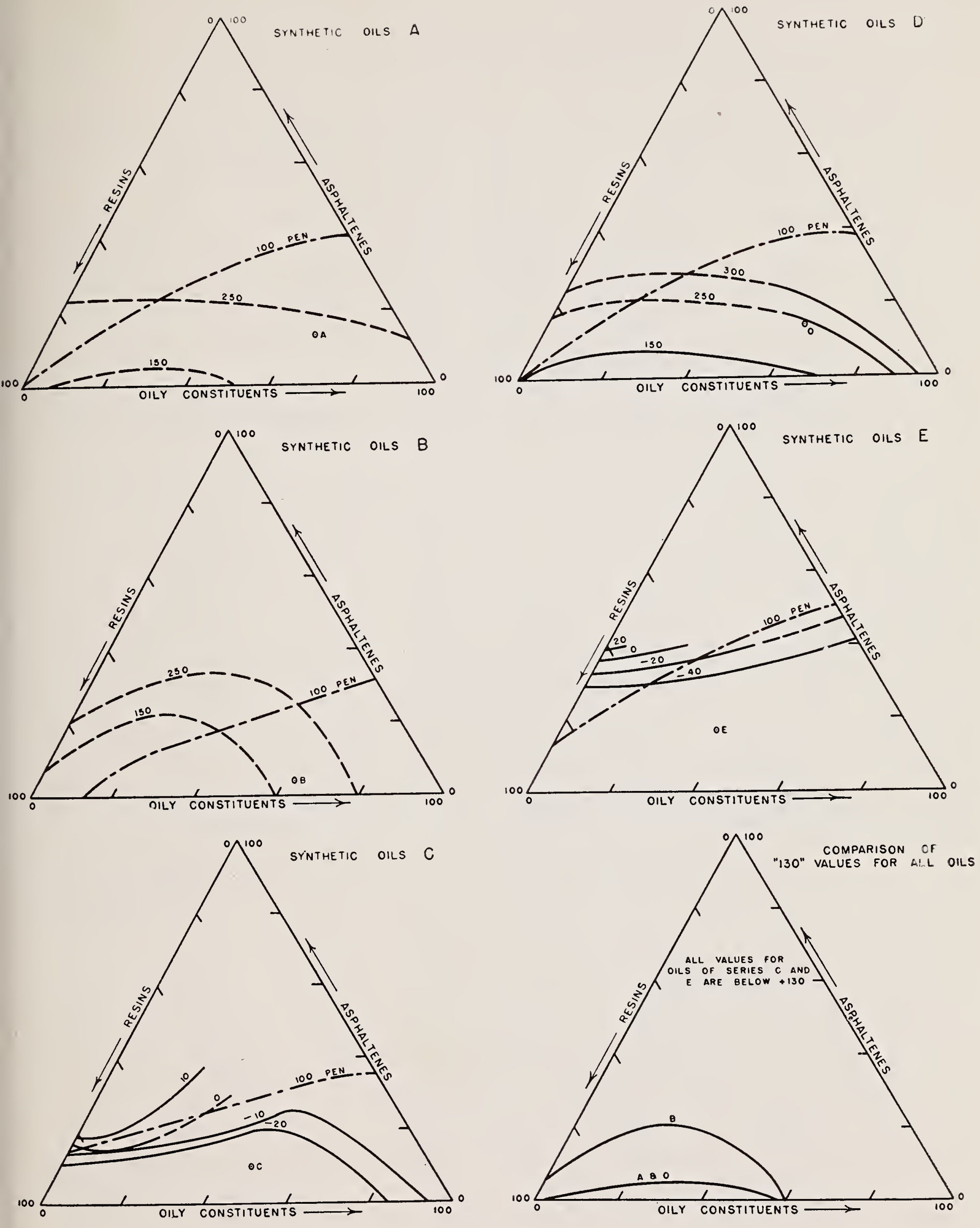
DUCTILITY IN CM AT 39.2° F, 5 CM PER MINUTE

FIGURE 17



PER CENT INSOLUBLE IN NAPHTHA

FIGURE 12



FLUIDITY FACTOR

FIGURE 18

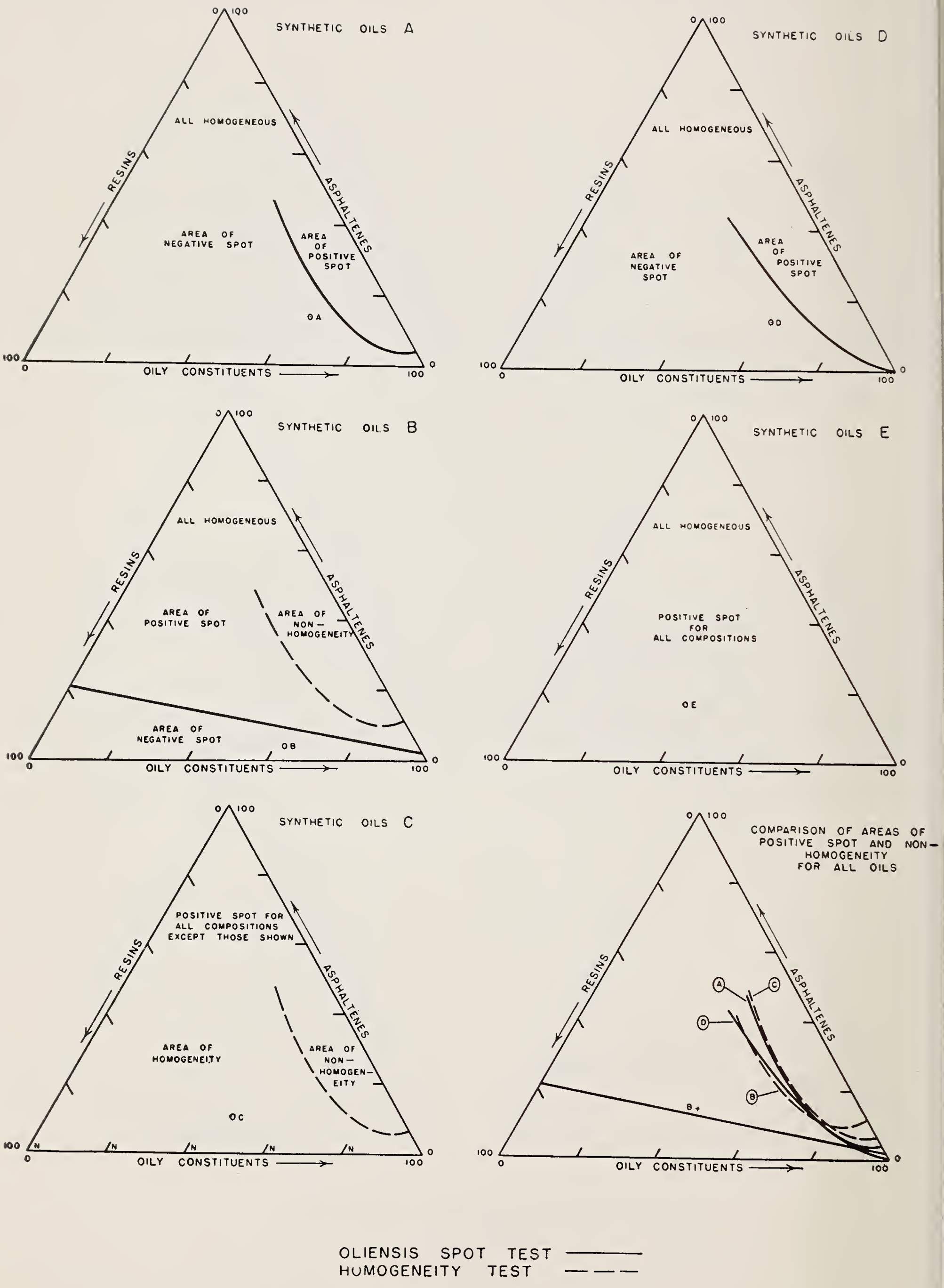
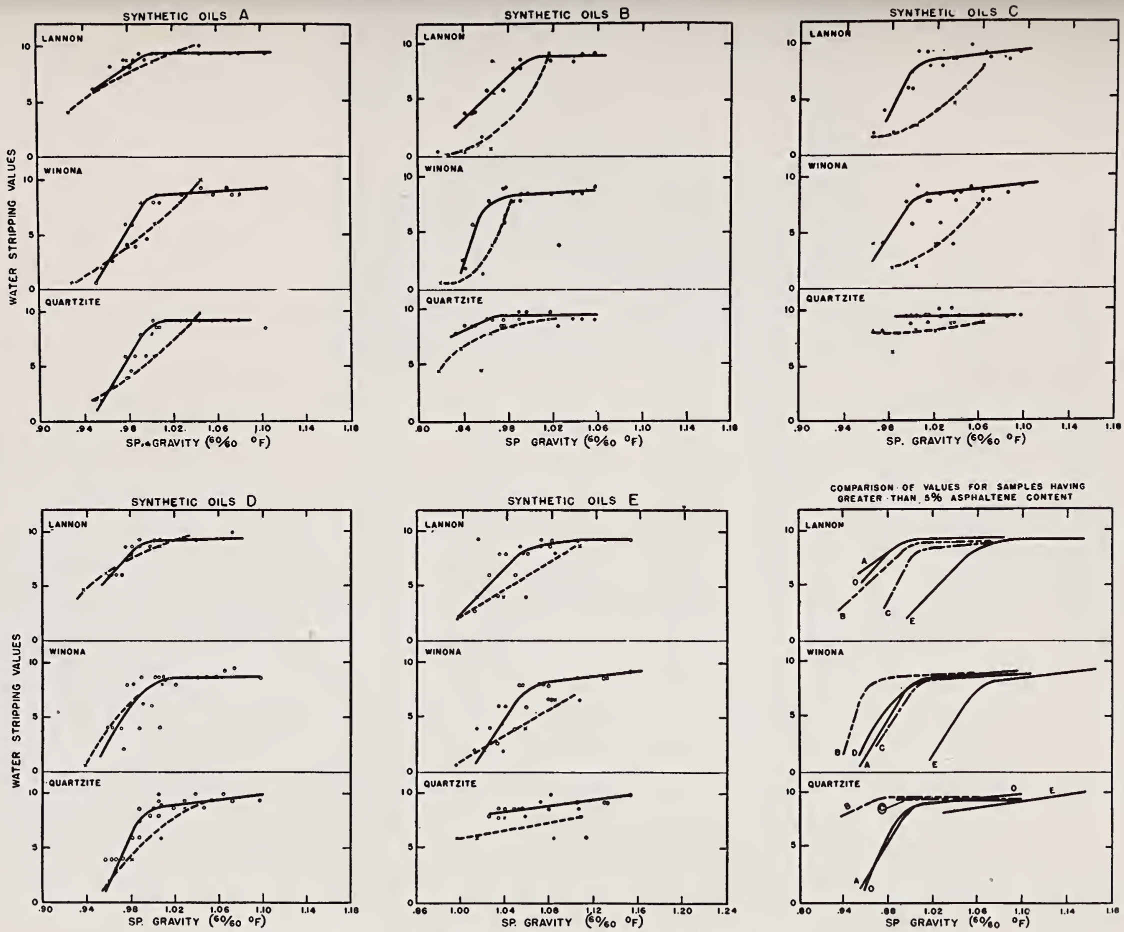

SYNTHETIC OILS A
ALL HOMOGENEOUS
AREA OF NEGATIVE SPOT
AREA OF POSITIVE SPOT
O A
RESINS
ASPHALTENES
OILY CONSTITUENTS
SYNTHETIC OILS B
ALL HOMOGENEOUS
AREA OF POSITIVE SPOT
AREA OF NON-HOMOGENEITY
AREA OF NEGATIVE SPOT
O B
SYNTHETIC OILS C
POSITIVE SPOT FOR ALL COMPOSITIONS EXCEPT THOSE SHOWN
AREA OF HOMOGENEITY
AREA OF NON-HOMOGENEITY
O C
N
SYNTHETIC OILS D
ALL HOMOGENEOUS
AREA OF NEGATIVE SPOT
AREA OF POSITIVE SPOT
O D
SYNTHETIC OILS E
ALL HOMOGENEOUS
POSITIVE SPOT FOR ALL COMPOSITIONS
O E
COMPARISON OF AREAS OF POSITIVE SPOT AND NON-HOMOGENEITY FOR ALL OILS
A
B
C
D
B+
0
100
OLIENSIS SPOT TEST
HOMOGENEITY TEST

FIGURE 11

FIGURE



WATER STRIPPING VALUES vs SPECIFIC GRAVITY AT $^{60}/_{60}$ °F

— — — 0% ASPHALTENE CONTENT IN SAMPLES (SAMPLES NO. 1 THRU 6)

——— GREATER THAN 5% ASPHALTENE CONTENT IN SAMPLES

FIGURE 26

	Road Oil A	Data from Diagrams Blended Oil A	Road Oil B	Data from Diagrams Blended Oil B	Road Oil C	Data from Diagrams Blended Oil C	Road Oil D	Data from Diagrams Blended Oil D	Road Oil E	Data from Diagrams Blended Oil E
Laboratory Composition										
Per Cent Asphaltenes	11.7		3.9		9.4		12.7		14.5	
Per Cent Resins	18.4		21.0		11.8		17.5		12.6	
Per Cent Oily Constituents	69.9		75.1		78.8		69.8		72.9	
Synthetic Composition										
Per Cent Asphaltenes	13.4		4.1		10.2		14.0		16.8	
Per Cent Resins	22.0		33.9		43.0		24.3		45.7	
Per Cent Oily Constituents	64.6		62.0		46.8		61.7		37.5	
Laboratory Composition from Curve										
Per Cent Asphaltenes		11.0		4.0		8.5		13.5		14.0
Per Cent Resins		17.0		23.0		10.9		18.0		16.0
Per Cent Oily Constituents		72.0		73.0		80.6		68.5		70.0
Specific Gravity 60° F./60° F.	0.989	0.985	0.959	0.956	1.029	1.030	0.996	0.991	1.079	1.080
Viscosity in Poises at 104° F.	64.2	40.0	42.7	35.0	82.0	30.0	67.5	60.0	102.7	60.0
at 122° F.	25.7	20.0	17.8	16.0	26.8	15.0	27.3	28.0	30.2	15.0
at 194° F.	1.7	1.3	1.4	1.3	1.2	0.8	1.9	3.0	1.0	1.0
Percentage 100 Penetration Residue	68.5	67.0	75.1	77.0	80.2	78.0	73.4	72.0	82.7	90.0
Ductility in Cm. (at 39.2° F.$_8$, 5 cm./min.)	9.5	7.5	8.0	6.5	0.0	1.0	8.5	8.0	0.0	0.0
Softening Point (R & B) In C.	49.0	50.0	48.5	50.0	45.0	......	51.0	52.0	40.0	40.5
Penetration of Residue	106		99		86		100		109	
Fluidity Factor	212.0	220.0	177.0	180.0	–54	–60	215.0	240.0	–85.0	–60.0
Viscosity Slope	21.2	23.0	22.7	23.0	28.0	26.0	23.6	23.0	30.4	29.0
Viscosity Temperature Susceptibility	0.8	0.79	0.78	0.77	0.942	0.95	0.829	0.82	1.032	
Asphalt Viscosity Index	7.5	7.7	8.5	8.5	8.8	8.0	7.4	7.8	9.7	9.0
Penetration Temperature Susceptibility	0.366	0.038	0.0386	0.036	0.0484	0.042	0.0347	0.035	0.0576	0.056
Per Cent Soluble In CCl_4	99.5	99.5	99.5	99.8	99.5	99.9	99.78	99.9	99.66	99.6
Per Cent Insoluble In Naphtha	14.5	14.0	5.0	5.0	10.74	12.0	15.76	15.0	21.0	20.0
Per Cent Loss by Evaporation	2.08	2.2	1.2	2.5	1.5	1.3	3.7	4.0	4.0	4.0
Oliensis Spot Test	Neg.	Neg.	Neg.	Neg.	Pos.	Pos.	Neg.	Neg.	Pos.	Pos.
Adhesion Test Data										
Water Stripping Data (Max. = 10)										
Lannon Stone (dolomitic)	8.7	8.0	5.0	3.0	8.0	8.7	8.0	8.3	8.7	8.5
Winona Gravel (granitic)	6.0	6.0	5.0	3.0	8.0	7.0	6.0	7.0	8.0	8.7
Quartzite	6.0	5.0	8.0	7.0	8.7	9.0	6.0	6.5	8.7	8.9
Riedel and Weber Test Data (Max. = 10)										
Lannon Stone	10.0	10.0	2.0	2.0	3.0	5.0	10.0	10.0	10.0	8.0
Winona Gravel	2.0	1.0	2.0	2.0	2.0	2.0	1.0	2.0	3.0	2.5
Quartzite	2.0	3.0	3.0	2.5	4.0	4.0	2.0	3.0	8.0	7.0

Efficacy of Oliensis Spot Test on Slightly Cracked Asphalts

By

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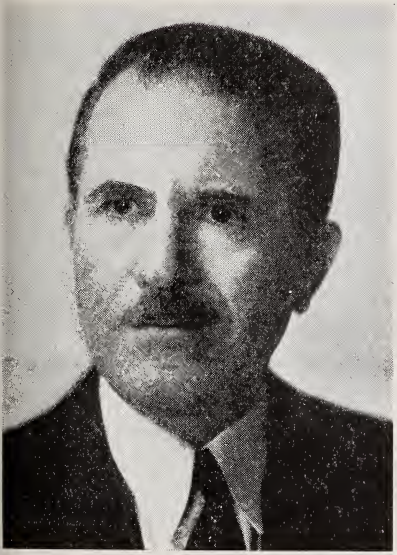


On page 85 of the 1938 Proceedings of the Montana National Bituminous Conference, the author read the following remarks by Mr. L. C. Krchma of Missouri:

> ''You can find materials (there is a recent paper being published by the Bureau of Mines) which are typical Negative Spot Test materials by the Oliensis test but have been cracked. Reducing the materials to asphalt, investigators are able to observe cracking which isn't noticed by the Oliensis test. It is difficult to classify materials, consequently, as cracked and uncracked materials on the basis of the Oliensis test.''

Correspondence by the author with the Bureau of Mines developed that Mr. Krchma's reference was to that Bureau's Report of Investigations 3435 entitled, ''Some Asphalts from Oregon Basin (Wyoming) Crude Oil,'' covering six high-vacuum distillations run in the laboratories of the Bureau's Petroleum Experiment Station at Laramie, Wyoming. The end products of these six distillations were, consequently, all vacuum residuals, and not cracking-still residues as might have been gathered from Mr. Krchma's remarks. However, in the case of two of the six residuals (namely No. 5 and more particularly No. 6) cracking had been suspected by the operators for the following reasons:

(a) The formation of a cloud or mist in the air condenser and receiver, and the condensation of a light distillate in the safety tube, had been observed in the latter stages of the distillation of these two;

(b) The simultaneous liberation of increased amounts of non-condensable gases had caused the absolute distilling pressure to rise; and

(c) Some of the test characteristics of these two residues had not been found consistent with those of the other residuals that had been produced in entirely normal distillation runs.

Yet in spite of the presumption of cracking raised by the foregoing circumstances in the case of both samples Nos. 5 and 6, these two residuals had proven to be Negative to the Spot Test, the same as the other four.

A careful analysis by the author of the comprehensive data found in R.I. 3435 did seem to warrant the belief that, at least in the case of Residual No. 6, some slight degree of cracking might have taken place; and the fact that nevertheless it had proven Negative to the Spot Test made him interested in determining whether the slight degree of overheating that was indicated by the data would not be revealed by certain refinements of the Spot Test that, for the sake of simplicity and ease of standardization, had been purposely excluded from the standard method generally in use, but which refinements should enable one to detect much milder degrees of overheating than the standard procedure could.

The Laramie Experiment Station very kindly sent the author, at his request, a 3-oz. sample of each of these six residuals, as well as of an additional residual, No. 7, that had been prepared subsequent to the release of R.I. 3435 but had been subjected to higher temperatures than the preceding six and did react Positive to the Spot Test.

The results of the author's investigations on these seven samples are given in full in his report of July 10, 1939, to the Laramie Experiment Station, which has since given him permission to abstract such portions from it as may be necessary for the exposition of this paper.

Manifestly, it was very necessary to decide what were reliable indices of cracking—not a very simple matter in view of the absence of a generally accepted criterion of what constitutes cracking. The mere formation of a cloud or mist in the condenser, the condensation of a light distillate in the safety tube, even the simultaneous increase in the vapor pressure, while all irregular, cannot necessarily be assumed to indicate a cracking of the residue, unless accompanied by other more positive evidence in the physical and chemical characteristics of the residue itself.

The method followed in R.I. 3435 for detecting abnormalties in the characteristics of the six residues has much to recommend it. It consists in drawing up a separate logarithmic-ruled graph for each of the individual physical and chemical characteristics reported—e. g. one for Specific Gravity, another for Softening Point, a third for Carbon Residue, etc., plotting out on each graph the test-figure obtained for that characteristic on each of the six residuals, the latter being arranged in the order of decreasing percentage-yield that each represents (i. e. roughly in the order of decreasing penetration), and then connecting with a straight line the points so obtained on the graph. Where any point is found to be sufficiently off the straight connecting line, an abnormality is of course indicated.

However, it is important to interpret these abnormalities with caution, for the following reasons:

In the first place, not every abnormality necessarily means cracking. For example, one of the criteria that is used in R.I. 3435 in respect to both residuals Nos. 5 and 6 to indicate that they have been cracked, is that those two residues have a softer penetration @ 77° F. than one would be warranted in expecting from the percentage-yield each represents. Granted this may be an abnormality for which a reason should be forthcoming; but unless a residue reflects, at least to some degree, each one of the characteristics typical of cracked residues from the same crude, and in addition shows at least some one characteristic that cannot possibly be explained in any other way than by cracking, to the sole fact that it appears to be softer than was to have been expected cannot be fairly attributed to cracking, and is than probably due to other and different reactions such as are at times encountered in straight distillations, among which may be cited, for example, refluxing, oxidation, etc.

Similarly, if this same residue, by virtue of its greater softness, also possesses a correspondingly lower Softening Point and lower Furols at elevated temperatures, such lower Softening Point and Furols manifestly cannot be attributed categorically to cracking if they are consistent with the characteristics either of perfectly normal residues of the same penetration or, for example, oxidized or refluxed residues of the same penetration.

Since, however, these various reactions alluded to here—cracking, oxidation, refluxing, etc.—affect the normal characteristics of a residual some in one direction and some in the opposite direction, there may frequently occur cases of partially-masked or partially-neutralized divergencies from the norm, whose true significance may for that reason be quite obscure. In such cases, it is manifestly safest to draw no inference at all from such particular divergencies from the norm, and instead depend for our diagnosis only on such other characteristics whose significance is so clear-cut or whose divergence is so pronouncedly in a given direction that its interpretation is unmistakable.

It is clear from the foregoing that the correct interpretation of divergencies from the normal is frequently no simple matter but requires both caution and careful weighting of the data available.

A direct clue, nevertheless, as to what criteria may most safely be depended on for diagnosing the presence of cracking can be obtained from the fact that if a cracked and a straight uncracked asphalt are both derived from the same crude and both are reduced to the same penetration @ 77° F., they will usually differ from each other in the following characteristics (for simplicity's sake only such tests are mentioned here as have been employed in R.I. 3435):

1. The Softening Point of the cracked asphalt will be lower than that of the uncracked of the same penetration @ 77° F.
2. The ratio of susceptibility of the cracked asphalt will be higher (that is, the ratio between its penetration @ 77° F. and that at

any higher or lower temperature, will be greater than the corresponding ratio shown by the straight uncracked asphalt).

3. The Furol Viscosity of the cracked asphalt at elevated temperatures will be lower than that of the uncracked of the same penetration @ 77° F.
4. The Specific Gravity of the cracked will be higher.
5. The Carbon Residue of the cracked will be higher.

The foregoing conclusions should aid materially in reaching a correct decision as to the extent of cracking, if any, present in the seven samples of residuals under discussion.

As to the first four of them (samples Nos. 1 to 4 inclusive), the history of the distillation of each is normal, and each of their physical and chemical characteristics is located in each case on or very close to the logarithmic norm of its respective graph. They have therefore all been classified in R.I. 3435 as normal, and their Negative Spot Test is therefore entirely consistent.

In the case of Residual No. 5, (35.3% residue, having a penetration of 19 @ 77° F.), cracking was assumed to have taken place because a mist or cloud was observed in the condenser at an oil temperature of 660-665° F., but since the distillation was completed at the same temperature, the period of misting must have been very short, and would certainly not justify in itself an assumption that actual cracking had taken place in the residue. It is therefore in order to determine what evidence can be found in the physical and chemical characteristics of Residual No. 5. From a study of the latter the following observations appear justified:

1. The Softening Point and the Furol Viscosity @ 210° F. and @ 275° F. are all slightly lower than would appear warranted in a 35.3% residue, but not lower than is to be expected from the actual penetration of the asphalt, namely 19 @ 77° F. Hence, as previously explained, an assumption of cracking cannot be justified from this group of three tests.
2. The Specific Gravity and the Carbon Residue of No. 5 are shown by Graphs 3 and 5 in R.I. 3435 to be exactly on the logarthmic line. These two characteristics therefore indicate rather strongly that no cracking is present.
3. The penetration of No. 5 at 32° F. and 60° F. (7 and 11 penetration respectively) appear abnormally high, even for the relatively high penetration of 19 at 77° F. Now in the first place, the fact that those two penetrations are high with respect to the penetration @ 77° F. indicates a lower ratio of susceptibility than the normal uncracked asphalt, and this in turn, as explained previously, points not to cracking but definitely away from it, since cracking usually results in a high ratio of susceptibility.

A careful analysis, however, of all the penetration figures given for No. 5 for the temperatures between 32° F. and 115° F. inclusive, particularly in comparison with the corresponding penetration figures given for its nearest counter-part (Residual No. 2), indicates that some error, typographical or otherwise, must have inadvertently crept into the recording of these two determinations at 32° F. and 60° F. for No. 5, and that these two determinations should be on the order of 5 and 7 respectively instead of 7 and 11 as reported. This can be rather clearly brought out by reducing the penetration figures for all temperatures to a common basis of 100 grams, 5 seconds (by dividing the penetration at 200 grams, 1 minute, by a factor of approximately 3, and multiplying the penetration at 50 grams, 5 seconds, by a factor of approximately 1.5), and plotting out the revised penetration figures so obtained on logarithmic-ruled paper. It is then seen that in the case of No. 2, and also in the case of No. 5 when the corrected figures are used, a substantially straight line is formed in each case by all the points on the chart; whereas if the original figures reported for No. 5 at 32° F. and 60° F. are employed as a basis, a zig-zag or ''Z'' line is formed with an abrupt break between 60° F. and 77° F., indicating a marked inconsistency as between the penetration figures reported for 60° F. and below, and those reported for 77° F. and above. This break is entirely removed when the corrected penetration figures of 5 and 7 are substituted in No. 5 at 32° F. and 60° F. respectively.

The correctness of the substituted figures of 5 and 7 was further corroborated by an actual check by two different operators in the author's laboratory, where both samples No. 2 and 5 were carefully remelted and penetrated alternately side by side at both 77° F. and 60° F.

If, therefore, instead of the penetration figures reported in R.I. 3435 for Asphalt No. 5 at 32° F. and 60° F., the figures 5 and 7 are plotted on Graph 8 of that report, the penetration of No. 5 at those two temperatures are on or so close to the logarithmic straight line for an asphalt of 19 penetration at 77° F. as to be within the limit of experimental error. Hence, no evidence of any abnormality (far less cracking) can be deduced against No. 5 from the penetration data.

The only conclusion that can therefore be justified in regard to Asphalt No. 5 is the one first given in the author's report of July 10, 1939, to the Laramie Experimental Station, to wit:

> ''There is found absolutely no evidence of cracking in Asphalt No. 5 in any of the physical or chemical test data available for that asphalt.''

CRACKING IN ASPHALT NO. 6

As to Asphalt No. 6, the Bureau report states that that was cracked more extensively than No. 5. There seems to be considerable data to support that statement. In the first place, the distillation of No. 5 was stopped at the same oil temperature as marked the first indication of misting in the condenser, and the maximum pressure recorded was 2.5 mm., whereas in the case of No. 6, while misting was observed also at 660-665° F. as in No. 5, the distillation was not stopped till it had attained an oil temperature of 690° F. and a maximum pressure of 40 mm. This indicates a somewhat harsher treatment than was given No. 5

The following three concluding paragraphs on No. 6 are quoted practically verbatim from the author's report of July 10, 1939, to the Laramie Experimental Station:

> "If a similar examination of the physical and chemical tests of No. 6 asphalt is made as in the case of No. 5, it is seen, first of all, that by extrapolation the penetration of Asphalt No. 6 @ 77° F. is about 325 or 350, which is approximately 1.5 times as high as the normally distilled Asphalt No. 3 brought down to the same Percentage Residue.
>
> "In view of this greater softness of No. 6, one cannot very safely draw any conclusions as to cracking either from the lowness of its Softening Point, or the lowness of its Furol Viscosity @ 210° F. and 275° F.; or from its high penetration figure @ 32° F. or 60° F.; since, as explained previously, these may all be largely or wholly the result of, and consistent with, its high penetration @ 77° F.
>
> "However, when one examines the Specific Gravity and the Fixed Carbon, one is struck by the fact that in spite of the greater softness of No. 6 (which ordinarily would warrant a slight lowering in these two constants) they are actually slightly but unmistakably higher than normal. This is a clear-cut trend towards cracking characteristics, and justifies the presumption that actual cracking, though cracking of but a slight degree, did occur in Sample No. 6."

CRACKING IN ASPHALT NO. 7

The evidence for cracking in No. 7, which had been prepared after the release of R.I. 3435, is still more clear-cut. In the course of its distillation, cracking in the condenser was observed at an oil temperature of 586° F., and the distillation was continued at a maximum pressure of 6.0 mm. to 721° F. oil temperature (135° F. higher than that at which cracking was first observed). A 29.7% residue was obtained, having a penetration @ 77° F. of 11 and a Softening Point of 160° F.

In the author's report previously referred to of July 10, 1939, to the Laramie Experiment Station, he added at this point the following comment on Asphalt No. 7:

> "The physical and chemical tests of this asphalt are not yet released, so no evidence is available as to the probability of its being cracked except two: (a) The high temperature to which it was carried beyond that at which the first misting in the condenser was observed, which in itself raises a presumption of cracking; and (b) the important corroborating fact that you found this residue tested Positive to the Spot Test, whereas you had found all other six asphalts tested Negative, which strongly indicates a much greater overheating in No. 7 than existed in any of the other six."

Fortunately, on August 14, the author received from the Laramie Station an advance tabulation of their physical and chemical tests of

No. 7, and it therefore became possible to determine whether the diagnosis of 'cracking'' previously reached for that asphalt would be borne out by the same trends in its characteristics as had been selected earlier in this report for proving both that No. 5 was uncracked and No. 6 was cracked.

The following tabulation answers this question. In it are given both the test data on No. 7 that had just been received from the Laramie Station, and the theoretically correct figures for a normal residue representing 29.65% of the original crude (as determined by the author's extrapolations from the same logarithmic charts that are shown in R.I. 3435):

CHARACTERISTICS OF RESIDUAL NO. 7 (29.65% RESIDUE)

	Laramie Laboratory Tests	Theoretical Tests For Normal 29.65% Residue
Penetration @ 77° F., 100 g., 5 sec.	10.7	4.4
Softening Point	160° F.	169° F.*
Saybolt Furol @ 210° F.	18,090	26,000
Saybolt Furol @ 275° F.	1,048	1,560
Penetration @ 32° F., 200 g., 60 sec.	3.1	1.3
Penetration @ 66° F., 100 g., 5 sec.	4.3	2.1
Specific Gravity 60/60° F.	1.064	1.058
Carbon Residue	30.2%	28.2%

*Based on prolongation of perfectly straight line through Softening Points of Samples 2 and 4 (instead of the curved line used in the R.I. 3435 graph).

To demonstrate the interesting parallelism between the trends revealed by the foregoing tests and those revealed by the tests of the other cracked asphalt (No. 6), the same sequence of thought, and as nearly as possible the self-same wording, will now be used in the next three paragraphs for a discussion of No. 7, as had been previously employed by the author in his report of July 10th in discussing the characteristics of No. 6. (A word-by-word comparison of the following comment with the corresponding three paragraphs previously devoted to No. 6, will bring out this remarkable parallelism still more clearly):

> If a similar examination of the physical and chemical tests of No. 7 asphalt is made to that in the case of No. 6, it is seen, first of all, that the penetration of Asphalt No. 7 @ 77° F. is 10.7, which is about 2.5 times as high as the penetration of 4.4 that is indicated by extrapolation for the normally distilled asphalt brought down to the same 29.65 percentage residue.
>
> In view of this greater softness of No. 7, one cannot very safely draw any conclusions as to cracking either from the lowness of its Softening Point, or the lowness of its Furol Viscosity @ 210° F. and 275° F., or from its high penetration @ 32° F. or 60° F.; since, as explained previously, these may all be largely or wholly the result of, and consistent with, its high penetration @ 77° F.
>
> However, when one examines the Specific Gravity and Fixed Carbon, one is struck by the fact that in spite of the greater soft-

ness of No. 7 (which ordinarily would warrant a slight lowering in these two constants) they are actually higher than normal, and to a greater extent than No. 6 was found to be. This is a clear-cut trend towards cracking characteristics, and justifies the presumption that cracking did occur in No. 7, and to a greater degree than in No. 6, thus confirming the diagnosis that had already been arrived at by the author previous to the release of the complete line of tests on No. 7.

OLIENSIS SPOT TEST ON SAMPLES NOS. 1 TO 7 INCLUSIVE

The procedure followed by the author in testing these seven samples was closely in accord with that outlined in the Appendix to his paper on "A Further Study of the Heterogeneity of Asphalt—A Quantitative Method," appearing in the 1936 Proceedings of the A.S.T.M., Vol. 36, Part II, page 506, with the exception that he employed the small round-bottom Soxhlet flask instead of the original Erlenmeyer flask. The naphtha he employed was Skellysolve "S," which he has personally been using as his standard for the last six years, and which in this case tested as follows:

A.P.I. @ 60° F.	49.4
Initial	300° F.
End Point	400° F.
Aniline Number	144.5° F.

Each of the seven small samples was first carefully remelted and stirred in its 3-oz. container and allowed to cool. The temperature of the glycerine bath in which solution was effected, and the time required to work each of the seven samples into solution was as follows:

Sample No.	Softening Point	Temperature Glycerine Bath	Spread Between Softening Point and Bath Temp.	Minutes Required for Solution
1	120° F.	255° F.	135° F.	8
2	130° F.	265-270° F.	135-140° F.	7
3	99° F.	230-235° F.	135° F.	8
4	111° F.	255° F.	144° F.	7
5	142° F.	275-280° F.	135° F.	7
6	93° F.	230-235° F.	139° F.	7
7	160° F.	300-310° F.	145° F.	8

As soon as the solutions had cooled to room temperature, and daily thereafter for the first two weeks (with the exception of the week-ends) and at longer intervals during the next two weeks, the solutions were thoroughly shaken up, stirred, and a drop was applied both on No. 50 Whatman filter paper and on clear clean glass. The purpose in employing the glass was to detect fainter evidence of carbenoids than is possible with the filter paper alone.

The author's motives in making the foregoing more or less minor amplifications of the standard Spot Test are foreshadowed in his original paper on the Spot Test, namely, "A Qualitative Test for Determining the Degree of Heterogeneity of Asphalts,'' A.S.T.M. Proceedings, Vol. 33, Part II, 1933, in which he states:

> ''When but small amounts of carbenoids are present they may fail to develop this characteristic rough nucleus on filter paper as clearly as on glass. Therefore the Spot Test on filter paper is not quite as delicate as on glass, though less subject to variations in personal equation . . . If doubt exists, it is well to set the solution aside in a tightly-sealed container, and test after thorough stirring, at intervals of about a week . . . If the haze becomes gradually more pronounced, it is evident that traces of carbenoids are present in the asphalt, since particles of this material, like those of most precipitates, tend to become larger on standing.''

It should be clear from the foregoing that when the test was first released, it was realized that by making the Spot Test on filter paper rather than on glass, and after aging only 24 hours instead of a longer period, a certain degree of delicacy that was inherent in the test was being sacrificed. This was done, however, in the interests of greater speed and ruggedness, and to make the test less subject to variations in personal equation. It follows that a very slight degree of heterogeneity was purposely ignored in the standard method, so that if a positive reaction did develop, it would be accepted without question by all parties as evidence of clear-cut heterogeneity, and so that the test might not be protested against as being too exacting, or too 'idealistic,'' to be practicable under ordinary refining conditions.

It also follows, however, that in research work or in special investigations, as in this case where two asphalts are suspected of being cracked and yet show no evidence of heterogeneity by the standard Spot Test, justification exists for going to the greater refinements rendered possible by the use of the glass plate and by the testing of the solution after longer intervals than 24 hours.

In fact, these refinements may in time perhaps be developed to serve a more concrete purpose: For while the present method of Xylene Equivalents published in 1936 constitutes a quantitative method of relative heterogeneity, there is as yet no quantitative method of relative homogeneity, since all asphalts that pass the test on filter paper in 24 hours must now be classified as equally homogeneous, whereas no doubt widely different degrees of homogeneity exist in asphalts. But a reasonably good method for the quantitative measurement of the degree of homogeneity may be ultimately worked out simply by determining the number of days or weeks or months that an asphalt-naphtha solution can remain standing at room temperature before developing evidence of heterogeneity. Work on this phase of the test is at present going on.

At any rate, the very full report found in R.I. 3435 on the numerous laboratory distillations of Wyoming crude, coupled with the close observations of the conditions prevailing during these various distillations, seemed to afford an especially good opportunity to test out the practical usefulness of these refinements in the Spot Test procedure for determining very slight degrees of overheating; and, as already stated, the seven naphtha solutions were accordingly tested not only on filter paper and at the end of 24 hours, but also on glass and periodically over a 4-week term. The following is a summary of the observations made:

Solutions 1 to 5—Samples Nos. 1 to 5 remained Negative throughout the entire period both on filter paper and on glass. This confirms the conclusion that they are all uncracked, and corroborates the interpretation given in the proceding pages of the physical and chemical data of No. 5, to the effect that the latter has not been cracked in the least.

Solution 6—No. 6, too, remained Negative for 48 hours on both glass and filter paper. At the end of the third day, however, while it still showed Negative on filter paper, it formed a very faint but definite nucleus on glass, just visible when the glass was placed at the proper angle against a dark background to highlight its center. Each day thereafter, this nucleus on glass grew slightly but definitely clearer. The first clear-cut evidence of a nucleus on filter paper, however, did not occur till the end of the seventh day, when a very faint shadowy ring appeared in the spot. At the end of the eighth day, the ring was still rather faint. At the end of the ninth day it was clearer still, and from the end of the tenth day to the beginning of the fifth week it has remained uniformly a sharply-defined but thin black ring within the brown spot.

After these spot tests had been observed for a period of about three full weeks, and the solutions showed evidence of having become fairly well stabilized, it was decided to start a check test on Solution No. 6, since its behavior had been so different from any of the others that it seemed advisable to determine how closely this behavior would be duplicated in a parallel test. A fresh solution was consequently made up, by a different operator, and daily spot tests were made of that too. Exactly as before, the solution remained homogeneous both on glass and on filter paper for 48 hours. Exactly as before, at the end of the third day, a very faint nucleus appeared on glass, but not on paper. Exactly as before, a faint ring first become discernible on paper at the end of the seventh day, grew stronger during the next two days, became sharply defined at the end of the ninth or tenth day, and became more or less stabilized thereafter.

This parallelism all along the line between the two solutions made from Asphalt No. 6 indicates this retarded manifestation of heterogeneity to be a definite property—probably as definite as any of the other constants of an asphalt; that this property is apparently reproducible within fairly close limits; and that it may ultimately become the means of developing a quantitative method for measuring relative homogeneity, to supplement the method of Xylene Equivalents already published for quantitatively measuring relative heterogeneity.

For the present, the Spot Test investigation of Asphalt No. 6 discloses that the heterogeneity of that asphalt is such that its carbenoids can be detected on glass after three days and on filter paper after seven days. While this indicates a definitely smaller degree of heterogeneity than could classify No. 6 as Positive within the meaning of the standard

Spot Test, it is neverthless definitely hererogeneous as compared with Asphalts 1 to 5 inclusive, and corroborates the existence of a slight cracking which was recognized in the analysis of its physical and chemical tests.

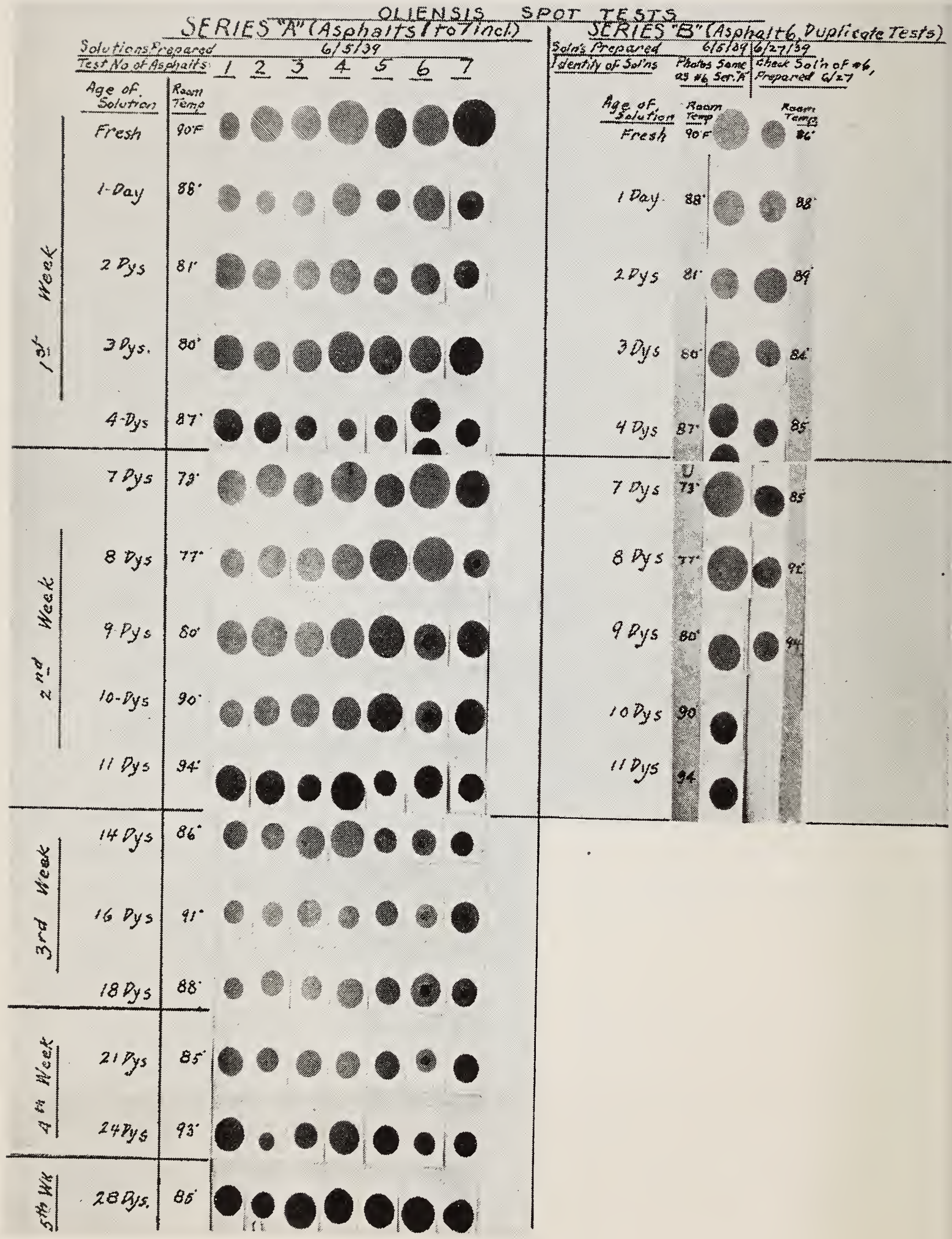
Solution 7—Solution 7 developed a heavy, black, annular nucleus from the first, and is the only one of the seven therefore, that tested Positive by the standard Spot Test. There is no doubt from that reaction that it is more heterogeneous and has been overheated to a greater degree than any of the other six asphalts, which is not surprising in view of the especially high temperatures to which it has been subjected.

PHOTOGRAPHS

The photographs annexed show, under Series A, the spots of the solutions of Asphalts 1 to 7 inclusive during the 4-week period of the investigation; and under Series B, a day-by-day comparison of the original and duplicate solution of No. 6. In studying them one cannot fail to be impressed (a) by the uniformly Negative spots of Solutions 1 to 5 inclusive throughout the month; (b) by the uniformily Positive spots of No. 7 throughout the same period; (c) by the slow transformation revealed in No. 6, which was Negative the first week, gradually developed an annular nucleus the first part of the second week, and stabilized at a thin, sharply-defined annular nucleus for the balance of the month; and (d) by the remarkable parallelism in the development and growth of the annular nucleus in No. 6 in both the original solution and the check solution, as illustrated under Series B.

CONCLUSIONS

The foregoing Spot Test investigation indicates that overheating is invariably detected by the Spot Test, but that since the standard method of procedure for that test has been devised purposely to ignore very slight degrees of overheating, the latter can only be detected by testing the standard solution after aging for a longer period than the standard 24 hours. This investigation therefore should serve to throw some light on the questions raised in Mr. Krchma's remarks at the last meeting of the Conference concerning the efficacy of the Spot Test.


OLIENSIS SPOT TESTS
SERIES "A" (Asphalts 1 to 7 incl.)
Solutions Prepared 6/5/39
Test No. of Asphalts: 1 2 3 4 5 6 7
Age of Solution
Room Temp
1st Week
Fresh 90°F
1-Day 88°
2 Dys 81°
3 Dys. 80°
4-Dys 87°
2nd Week
7 Dys 73°
8 Dys 77°
9 Dys 80°
10-Dys 90°
11 Dys 94°
3rd Week
14 Dys 86°
16 Dys 91°
18 Dys 88°
4th Week
21 Dys 85°
24 Dys 93°
5th Wk
28 Dys. 85°
SERIES "B" (Asphalt 6, Duplicate Tests)
Sol'n's Prepared 6/5/39 6/27/39
Identity of Sol'ns
Photos Same as #6 Ser. "A"
Check Sol'n of #6, Prepared 6/27
Age of Solution
Room Temp
Room Temp
Fresh 90°F 86°
1 Day 88° 88°
2 Dys 81° 89°
3 Dys 80° 84°
4 Dys 87° 85
7 Dys 73° 85
8 Dys 77° 92°
9 Dys 80° 94
10 Dys 90
11 Dys 94

The Measurement of Mechanical Stability of Bituminous Mixtures and its Correlation With Service Behavior

By

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INTRODUCTION

As in the case of other materials, the producer of asphalts has a direct interest in the quality of the products which are made by combining his materials with others, and of course also has a better opportunity to investigate his own materials than has anyone else.

In the case of asphalt, we are dealing with a material which by itself seldom has value for anything; therefore investigation of its "qualities" is nearly always investigation of the quality of its combinations. Thus investigation of the quality of road asphalts simply means investigation of the quality of asphaltic road mixtures. The circumstances indicate a somewhat different approach on the part of a producer's engineer from that which is usual in the case of public works road engineer. The latter usually has before him limited sources of asphalts and limited sources of road aggregates. Provided he has some kind of test for mixtures, which has been correlated by experience with the quality of road carpets, it is possible for him to arrive experimentally at some of the best combinations possible with the materials available. Excellent results are often obtainable when this procedure can be supplemented by wide personal experience.

This course is impossible for the producer, as no laboratory could in the course of a lifetime work over by trial and error all the combinations possible with the product. He thus is inclined to take as direct a route as possible to fundamental principles, and this requires highly analytic methods. In other words, to know that a certain combination is better or worse than another is not enough; he has to know why, in terms of its separate components. This sometimes requires considerable investigation of materials beyond his own product, but is unavoidable.

The indicated objective in this field is the development of the best combinations possible with typical materials available. It may or may not be possible ultimately to discover a "best" asphalt or aggregate. Experience and research to date indicate that the widest and most profitable field will be discovery of optimum combinations and improvement of engineering practice. A great range of types of both asphalts and aggregates have been employed with equal success, and the best contribution to continued lowering of costs seems to be the extension of this form of achievement.

This paper of course deals with only a small part of the whole field of research on road mixtures.

The method principally discussed is as yet a matter of analytic research and not of routine testing for quality.

(1) FACTORS OF RESISTANCE AGAINST DEFORMATION

While it is possible to test bituminous mixtures under such conditions that the strength of the individual aggregate particles becomes a factor, the present analysis is confined to conditions of normal or high temperatures, and to relatively low testing speeds, where such strength is not a factor, internal movement of the material here involving the motion of mineral particles past or over one another without individual rupture.

Under this condition the resistances to be expected are:

1. Interlock (mutual interference of the mineral particles with one another).
2. Internal friction within the aggregate, between its surfaces.
3. Cohesion due to the asphalt.
4. Apparent cohesion due to surface tension effects in asphalt, etc.

Some discussion of the above factors follows:

1. **Interlock:** In order for the particles to oppose resistance to the movement of one another, it is necessary that such resistance be transferred from one particle to another until it reaches a resisting medium external to the body of material in question. This external resistance may be imposed by a buttress or through friction upon a surface. The exact nature of the ultimate transfer of resistances has not yet been entirely determined. The movement necessary to overcome interlock also usually involves a rolling motion of the particles upon one another, which causes the material to tend to expand. In some cases this expansion is counteracted by the internal motion of the material causing consolidation. This expansion or dilation causes a certain effect on the apparent unit stresses due to difference in volume and hence in the surface to which the stress is applied. Ordinarly, however, the amount of this is small.*

2. **Friction:** We may say that a resistance is frictional when it is directly proportional to the load applied. Under the classic definition, friction arises between two surfaces which move over one another without having their roughness characteristics unchanged. If, however, any resist-

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*It is a moot question whether a true interlock can be said to exist after dilation is complete, i. e., while the material is deforming under constant conditions. This will be further discussed.

ance arises which has a statistically constant proportion to the load applied, but which also involves changes in the sliding surfaces, it cannot ordinarily be distinguished from true friction. Such a case may arise where a point or edge of one particle grooves another at a resistance proportional to the load. There is an indication that such effects exist in measurements of internal friction, in the fact that the apparent coefficient of internal friction is usually affected by the degree of consolidation of the granular material. However this may be due also to other effects later discussed.

3. **Cohesion:** Arises from the individual molecules of the binder clinging to one another and in turn to the aggregate. In this connection it should be recognized that asphalts in thin films such as occur in bituminous mixes exhibit different properties from those found in the same asphalt in a larger body, such for instance as a ductility briquette. This resistance factor involves quite a complex physico-chemical problem, especially in relation to the affinity of the binder molecules for the mineral surfaces. It appears probable that a mixture which shows a high anti-stripping quality in the presence of water will also show high cohesion under dry conditions.

4. **Apparent Cohesion:** Housel believes that the introduction of the conception of apparent cohesion causes an unnecessary complication of the theory, and that cohesion and apparent cohesion can be considered for practical purposes as the same pheomenon. We would say that from the point of view of the road engineer this is probably true, but from the point of view of fundamental research on the role of the asphalt, the possibility of separating and measuring apparent cohesion is of interest. For simplification, however, we will use only the term ''Cohesion'' in the discussion, as inclusive of both terms.

(2) THE FUNCTION OF MECHANICAL TESTS

As with other materials, tests can be classed as those made by the application of compressive loads, those made by shearing loads, and those made by tensile loads.

Fundamentally, however, there is no such thing as a compressive test except in the yield of volume compressibility. The compression tests as ordinarily used are simply methods of applying shear or flow tests, the material always failing in this manner only.

In considering the practical application of tests, it is necessary first of all to consider their relation to the manner in which loads are applied in practice. Under traffic the load is always ''compressive'' in a non-elastic material. That is, if the material moves, it moves by shear or flow. For evaluating resistance to such stresses, shear tests, either those usually so termed, or indirectly applied as by ''compression'' tests, are obviously called for. The compression test has a certain appeal here from its apparent likeness to the condition of loading on the road. This likeness, however, is not at all complete, for the following reasons:

(a) Under road conditions the loaded prism is subject to side pressure from the adjoining material.

(b) The normal test method involves pressure between rigid surfaces. Ordinary road bases exhibit some resilient resistance, or sometimes move under load by consolidation or settlement of underlying layers, while the tire through which the load is applied causes peculiar stress distributions on the loaded area which are unlike those under a rigid plate.

Obviously we are not yet in position to master all these factors at once, but must try to get at the fundamental properties of the material and later endeavor to correlate them with the influence of the surrounding conditions. For that reason, at the beginning, logic calls for as complete elimination of surrounding influences as possible. In this respect some form of compression test, which is an indirect shear test, appears superior to shear box or shear cylinder methods, since it can be made under conditions where the shear planes are freer from the influence of adjacent equipment, and especially are free from edge effects and the transmission of forces to the grids such as exists in the shear box. Moreover, in practice the material is largely free to form its own flow lines by the principle of least resistance. In the shear box or shear cylinders the attempt is to define the shear plane as closely as possible. Where the shear area is very large as compared with the particle size, as in using a large box or cylinder with sand, this close definition is not a misleading factor. Where gravels are in question, it is impracticable to construct a shear box large enough to have the same relation.

In the shear box also we find a definite edge effect, as illustrated by the fact that as we increase the size of the box, using the same material, the apparent coefficient of friction decreases. To date we have not seen results published by anyone who has increased a shear box to such size that results become constant. In compression tests it is possible to arrive at constant conditions quite easily. Normally this is reached in bituminous specimens of any diameter at height/diameter ratio of 2 or over.

The principal defect of the standard compression test is that it is not analytic. It will show how much load a specimen will carry under given conditions, but it will not show why it will carry it. It will yield such information only through a number of tests under rather elaborate variation of conditions.

The Hubbard-Field stability test is essentially a flow test in which a portion of the material is extruded through an orifice. We are not prepared to discuss the usefulness of this test in relation to empirical correlations with practical results as derived from experience. We can say, however, that we have been unable to employ it for purposes of direct analytic study because of the very complex resistance phenomena which it produces. These phenomena appear impossible to analyze mathematically, and we find our best hope in its connection to be the study of a large number of correlative tests.

The usefulness of a tensile test appears to be found applicable only to certain conditions, (a) in the case of a bituminous material of elastic properties and therefore subject to slab stress distributions to the subgrade, and (b), in the case of temperature stresses. Since both these conditions are likely to exist at relatively low temperatures, the test appears to be essentially one applicable to such conditions, and cannot be at the present time analytically related to stability requirements under usual conditions.

By process of elimination, our European associates have adopted in the Amsterdam Laboratory, the triaxial or pressure cell test with Mohr circles. Equipment suitable for the use of this test was developed in more than one quarter during the past few years, but so far as we know principally by Dr. Buisman of Delft University, Ehrenburg at Berlin, Terzaghi at Vienna, and F. N. Hveem of the California State Highway in this country, though the method of use by the last is different from the others.

This method not only applies stresses under conditions which have a closer parellel with actual conditions than most other methods, but under the Mohr Circle theory gives an analysis of the behavior of the material as well as the gross result.

(3) THE PRINCIPLE OF THE TRIAXIAL TEST

The principle of the test is closely related to that of the compression test, and in fact we have found that the results of the former can be converted into those of the latter (but not vice versa).

Under the classic theory of the compression test, the material deforms by shearing along a series of parallel planes, the angle of which with the vertical, θ, is regarded as characteristic of the material, and as having such relation with the internal angle of friction that $\theta = 45° - \phi/2$, where ϕ is the angle of friction. Figure 1A shows a single one of such ideal planes. If the material shears along an indefinite number of planes, and if the influence exerted by the base is the same as that of the plunger, a fuller diagram gives Figure 1B. The net movement of the particles due to this combination of shear or flow planes is a horizontal flow which causes a barrel-shaped deformation as in Figure 1C.

If the material is rigid, or very non-uniform in its composition, localized shear planes will develop as in the case of cement concrete. Also, in the ordinary compression test, if deformation is carried far enough, axial splitting takes place, thus introducing a certain tensile factor. In using the triaxial test with normally well-proportioned materials, we seldom find any deviations from the typical barrel-shaped form, and no splitting.

The difference between the compression test and the triaxial, mechanically speaking, is that in the latter, during deformation, a lateral hydraulic pressure is maintained against the sides of the specimen. This is transmitted through the specimen and results in an increase in the vertical load necessary to deform. The mathematical relation between the horizontal and vertical pressures under this condition is found in the Coulomb formula, which has been largely used to calculate pressures against retaining walls. Three principal stresses exist as follows:

P1 is the vertical load per square inch.

P2 is the load upon the shear plane per square inch.(Normal pressure).

P3 is the hydraulic pressure per square inch against the side of the specimen.

These are shown in Figure 1D.

The development of the mathematical formulas is beyond the scope of this discussion, but we consider them as being sound because of the consistency of results derived from their use.

The relations of the Coulomb formula are as follows:

$$P3 = P1 \tan^2 \theta.$$

P2 tan ϕ is the frictional shear resistance, along the shear plane.

$\dfrac{P1 - P3.}{2}$.. cos ϕ is the shearing stress at failure.

If the material has a constant resistance along the shear plane, the total resistance at any load will be the constant resistance, C, plus the frictional resistance, or shear resistance (sr) will be C plus P2 tan ϕ.

In other words the resistance of the material consists of two factors, frictional resistance due to the load itself and a constant resistance inherent in the material regardless of load.

The Mohr Circle method is able to separate these factors for examination.

The stresses due to an applied load can be represented as in Figure 2.

In Figure 2A the stress relations on a plane which has the slope with the vertical of a are shown, Pa being the pressure normal to the plane, with P1 and P3 laid off on the horizontal axis. In this figure a is greater than θ, and the shear stress, cb, is less than the frictional shear resistance. Thus the material cannot fail along this plane in free deformation.

In Figure 2B the relationships are such that c'b' equals $\dfrac{P1 - P3}{2}$. cos ϕ

and shear stress is equal to shear resistance. This is, of course, the condition of constant plastic flow. If as in Figure 2C a tangent is drawn to the circle at b', it will pass through the origin, under the relationships of the formula.

For a material which has no resistance other than frictional, the solution is complete with the drawing of a single circle. The next step in arriving at the characteristics of a material having other resistances is shown in Figure 2D, where since the relationships of P1, P2, and P3 are geometrically constant, any circle drawn at any value of P1 which may be selected, will be tangent to the same line, which is called the ''Mohr envelope.''

If in addition to the frictional resistance, a constant resistance exists, the result will be to increase the vertical pressure more than the horizontal by a constant value, thus increasing the size of all circles but leaving the slope of the envelope the same. The resultant intercept on the vertical axis is the value of the constant resistance on the shear plane, and if at any point of tangency we draw an ordinate to the horizontal axis, its value will be the total resistance on the shear plane for the load represented by the P1 value for the circle in question, and it will be equal to C plus P2 tan ϕ.

The compressive test can now be related to the results, because in t the point where the stress deformation diagram gives a horizontal angent, which represents the compressive resistance measured vertically, s the limiting case of the Mohr circle. That is, it represents a Mohr circle where P3 is zero and the circle is tangent to both the vertical axis and the Mohr envelope. This case is shown in Figure 2F, where Pm is the compressive strength as calculated.

(4) MECHANICAL DETAILS OF THE TEST

The equipment used, which varies greatly in details but always embodies the same principle, consists of an outer pressure cylinder in which is placed the specimen enclosed in a waterproof membrance which offers no appreciable resistance to deformation of the material. A vertical load s then applied to the top of the specimen by a compression machine, and a horizontal pressure through liquid in the pressure cylinder. In taking each Mohr circle, the speed of deformation is constant, as is also the temperature in case of bituminous materials. For each point the material s deformed until P1 and P3 are in equilibrium, this equilibrium being secured by bleeding off the liquid in proportion to the deformation.

Causes of difficulties which need to be guarded against are:

Use of too resistant a rubber membrane.

Friction between the plunger and the outer cylinder opening.

Tension in the rubber cylinder due to deformation of the material.

Changes of temperature.

Judging that the pressures have reached equilibrium too quickly.

Changes in testing speed.

(5) REDUCTION OF RESULTS TO STATIC CONDITIONS

For fundamental analysis of the action of a material under all practical conditions, it is necessary ultimately to cover the range of conditions from static loading to sharp impact. The triaxial method is very near the former conditions, being essentially a slow speed test, and in this respect is exceeded only by the constant load method, with light loading, used by the British Ministry of Transport. For fundamental examination of the roles played by the aggregate and the asphalt, it is desirable to reach as nearly static conditions as possible. With the Mohr circle method, this is accomplished by repeating the test at various speeds. It is found in this case that the slope of the ϕ-line will remain constant, but that the Mohr constant will vary with the speed. Points can then be plotted for the constant at the various speeds which are connected with a line passing through the zero-speed axis.

(6) FRICTIONAL CHARACTERISTICS OF AGGREGATES

In actually using the test it is found that many interesting and illuminating phenomena appear.

We have been endeavoring to relate the qualities of the aggregate to those of the mixture.

Certain characteristic actions of the aggregates had already been known by previous research, for instance that the coefficient of friction in the material increases with degree of compaction of the specimen, and that in shear tests it decreases with the total load applied. The two phenomena may be somewhat related.

The decrease with increase of applied load is apparently due to greater smoothing of the surfaces in moving over one another; but we think it advisable to distinguish theoretically between a decrease of friction which appears almost immediately with application of load, and one which appears gradually with increasing deformation. The two can be partly distinguished in the Mohr circle diagram.

If a series of diagrams with the same material are taken at various pressure ranges, it will be found that the higher ranges show decreased friction, and of course an apparent increase in interlock because the envelope when projected intersects the zero axis at a higher point.

The choice of pressures to be used is thus important. In road work the actual pressures on the material are in excess of those used in shear box tests in soil mechanics, and the material thus behaves differently.

The actual pressures developed under a standing tire are in the order of 40 to 200 lbs. per square inch locally, and may sometimes be considerably greater.* The Mohr circle diagram of course has to be taken at pressures in excess of the compressive strength of the material, but if taken too far in excess, will not represent the action under load. In comparing the results of tests on bituminous mixes of various strengths, we found this quite noticeable. Current Amsterdam investigations employ a maximum pressure of about 400 lbs. per square inch. We have done considerable work at a maximum of 1000 and are relating this to lower stresses, in general overlapping normal soil mechanics work ranges for our lowest stresses.

The results of tests on Ottawa sand, a fine paving sand, 10 mesh, and a ⅝″ graded gravel aggregate, are shown in Figure 3. It is evident that the proper selection of pressure ranges is very important, especially since the critical changes appear to lie in the pressure range where traffic loads are applied. It seems that the pressure resistance of the aggregate particles begins to be important at a lower range of pressures than might be supposed. However, we believe that distinction should be made between shear resistance to breakage and resistance to smoothing of surfaces, as the latter seems to appear much earlier than the former.†

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*Impact ratios have been found up to 2.25.

We believe that the higher frictions found with greater consolidation of the material in an original specimen may be due to the fact that in a compact specimen greater areas are in contact and the unit load during movement on the contact surfaces is reduced, thus producing less tendency to smooth down the surfaces. Higher densities may also produce greater interlocks which appear as 'friction.'

In comparing these results with soil mechanics experiments we find similar results. Massachusetts Institute of Technology, for instance, shows a decrease of about 5 to 6 degrees in the value of ϕ in going from 7 lbs. vertical load per square inch to 112, with shear box. Actual values varied from 27 to 36 degrees according to degree of consolidation.*

The results shown in Figure 3 were obtained with several different specimens at the different pressure ranges, and not by continuous deformation of single specimens; the latter method would probably give a somewhat greater decrease in angle of friction and give an indication of the extent of movement required to produce noticeable decrease in the angle of friction. High apparent values of ϕ were obtained probably because the height/diameter ratio in this case (approximately 2) was still too low to eliminate end-effects.**

It will be noted that these curves enable one to distinguish to a large degree between the initial effect of application of loads of different magnitudes, and the effect of progressive deformation under loads of each magnitude. The shape of the curves indicates that under the heavier loads the decrease in coefficient of friction occurs largely by the time the first circle is taken. (Three circles were taken for each sample here.)

These factors appear to be of importance in considering the effect of rolling, and lend some point to the statement sometimes heard that over-rolling can damage a pavement.

It is noticeable that in the materials shown the most critical region lies below 300 lbs. load per square inch.

(7) THE RELATION BETWEEN DRY AND BITUMEN BOUND AGGREGATES

It would obviously be desirable to be able to test a dry mineral aggregate and predict its behavior with a given asphalt from the results. Results so far indicate, however, that this problem should be approached with caution. The general relationship is now well established to be that the introduction of bitumen results in large increase in the Mohr constant and some decrease in angle of friction. Moreover in general the harder the asphalt the greater the increase in C and the greater the decrease in ϕ.

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†However, we find indications that reduction of friction may be due also to other factors than smoothing.

*Standard Ottawa sand.

**These figures were constructed on the more usual assumption that interlock does not exist during movement.

However, so far as we have gone, the relation between the values of C and ϕ in dry aggregate and in the bituminous mixture is not well enough established for practical predictions. For one thing, C in the dry material is of quite low magnitude unless derived at rather high pressures. Under the test conditions of results discussed here, the Mohr envelope for dry materials was normally curved, and passed through the origin. Hence any "C" value, which is found by drawing a tangent to such an envelope, is apparent rather than real, depending largely on the applied pressure.

Experiments in Amsterdam have not usually shown the apparent C of sands and gravels to exceed 1 lb. per sq. inch, at height/diameter ratio of 3, the envelope in that case being nearly straight. The work in Amsterdam has accumulated a large amount of interesting and significant information on the interrelationship between the aggregate and the bitumen.

A case from our own experiments is given below:

Aggregate	C for agg.	ϕ for agg.	C for mix	ϕ for mix
A	43*	38.5	50	31.0
B	62*	39.3	101	29.3

The values here were not extrapolated to static conditions but were taken at a given testing speed. Temperature was 77° F. Vertical pressures used were 500, 750, 1000, corrected for deformation. Stabilities determined at optimum oil contents, for the same oil (50/60 California asphalt).

The grading curves for the two materials were nearly alike, but sample B showed angular particles and gritty texture.

The differences in the aggregates are thus exaggerated in the mixes, except that the frictional characteristics are slightly reversed. The indication here is that an angular nature of the aggregate may result in better contacts of surfaces, etc., thus giving better cohesion in the bound material. However, our work in this direction has not gone far enough to draw many conclusions as yet.

In general the complexities of behavior of the material as exposed by the triaxial test are such that we are not as yet prepared to recommend the method for routine determinations. It may be that its chief value in the future will develop along the line of interpreting and correlating simpler tests.

However, we believe that its use, in addition to its primary purpose, has already thrown some light on the fundamentals of behavior of the aggregate under traffic. Since previous investigators have discovered that aggregates tend to degrade somewhat under traffic, and since indications of a breakdown of coefficients of friction under loading have been established, the general conception seems to be as follows:

1. Under rolling and later under traffic, particles break down somewhat. This results unfavorably in reducing the size and perhaps the stability due to large particles; but favorably in

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*High apparent values due to high pressures used.

producing closer grading and more angular particles and flat faces.

2. Under rolling and later traffic, coefficients of friction decrease somewhat. This results in some instability but facilitates continued compaction. Most of this decrease may occur early.
3. Closer grading and increasing compaction due to the above effects produce a greater area of adhered surfaces between particles, thus increasing the stability due to the bitumen. (Unless it also fills the voids and unstabilizes the mix due to excess of bitumen.)

However, although the above phenomena are actions of the aggregate, they are so affected and modified by the nature of the asphalt that they cannot be considered separately from it in the total mixture.

(8) THE EFFECT OF HEIGHT-DIAMETER RATIOS

The problem of correlating the results of laboratory tests with field conditions involves approximating the corresponding specimen dimensions. In the use of either the compression or triaxial test it is well established that the height-diameter ratio should be at least 2, and preferably 3, if distortion of the shear planes is to be avoided. Such distortion is also accompanied with the effect of the coefficient of friction between the base plate and plunger, which is likely to be highly variable. In experiments at Amsterdam, the use of a rubber disk on the ends of low height specimens resulted in marked decrease of apparent compressive strength and greater reproductibility. We have found that in running Mohr circle tests at low height-diameter ratios, the angle of friction remains relatively constant with decreasing H/D ratio until the ratio of 1 is approached. The apparent friction then shows an increase which becomes quite sharp below H/D of 1, and in some cases tends nearly to reach infinity between 0.5 and 0.25, values tending to spread greatly. The apparent C values show such peculiar curves that we are not yet prepared to discuss them, but in general it is noticed that the whole Mohr diagram becomes distorted in its relationships*. Whether the distortion can be reduced to mathematical factors is a question for the future. For the present we wish to emphasize that the H/D ratio for Mohr circle tests should be kept at least above 2 if the true characteristics of the material are to be evaluated.

The relation between those characteristics and practical H/D ratios is a separate problem.

In practice modern tires give a surface about 5 inches wide, as a minimum; and load is seldom applied on pavements over 6 inches thick. As a result of collected information and observation, Hveem uses an H/D ratio of 0.6 in stabilometer tests. This would correspond to a modern wheel tread applied on a 3-inch pavement. While the correlation with practice is logical, unfortunately this is just in the region where the distortion of analytic tests becomes very serious and also rather erratic, again emphasizing the fact that analysis of fundamental qualities must be considered as a problem precedent to correlation.

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*If it is assumed that no interlock exists, decreasing H/D ratio seems to result. Change of ϕ much earlier.

It appears then that in this relationship the apparent action of the pavement begins to be seriously affected by the qualities of the base material, and that these comprise something more than can be shown by bearing and penetration tests.

We may have at one extreme a rigid base approaching the effect of steel plate, and at the other a resilient or spongy soil approaching the effect of a rubber disk. Again, the degree of friction and/or adhesion between the pavement and the base will effect the flow qualities of the pavement, and so will the degree of cohesion within the base itself. This is a field which remains to be covered.

(9) THE EFFECT OF TEMPERATURES

It is of course well known that the fluidity of some asphalts is more susceptible to temperature change than others, and the degree of such effect is rather easily determined in bituminous mixes by stability tests. In practice it has been found that satisfactory pavements can be made with asphalts of all the normal temperature-susceptibility types, granted only that due attention is paid to the factor of aggregates, asphalt content, etc.

In the field of testing, however, unless we are to run a series of tests at different temperatures on each material, which would be difficult and expensive as a routine matter, there remains the problem of selecting the most representative testing temperature. That is to say, does most deformation take place in the relatively short period of hot weather, or in the longer period of moderate temperatures? Some consider that the worst condition should be reproduced, and thus test at the highest summer temperatures normally to be expected. However, it should be recognized that if all the asphalts being dealt with are of the same temperature-susceptibility qualities, the magnitude of the problem is reduced somewhat. We think that the situation points somewhat to an adjustment of testing methods to the climate in which the material is used.

(10) SPEED EFFECTS

The influence of testing speed upon a bituminous mix specimen is of course very vital, and a material which shows purely plastic flow under the methods heretofore described will exhibit fracture, elasticity, brittleness, etc., at higher speeds. How far the conditions existing under road traffic are approached by increasing the testing speed is another question of the future. In considering this, it should be borne in mind that two distinct differences from static methods exist: (a) the effect of a very brief duration of pressure, and (b) impact effects which increase the normal load.

If the pavement is perfectly smooth and no vibration exists within the machine itself, the effect is simply (a). Under normal conditions, irregularities of pavement and oscillations of springs add the factor (b) to (a).

Unquestionably both these factors present a picture widely different from that which can be shown by static tests. The fact, however, that investigators using quasi-static tests (such as Hubbard, Hveem, Vokac, and others) have found parallelisms between their tests and observed field results, to such an extent that considerable betterment in quality has been obtained through them, indicates that connecting links exist and can be quantitatively investigated.

SUMMARY

1. **The Factors of Internal Resistance in a Bituminous Mixture**
   a. Interlock of particles.
   b. Friction between particles.
   c. Cohesion and apparent cohesion.
2. **The Function of Mechanical Tests**
   a. The greater part of the loading in practice is applied compressively.
   b. Tensile stresses are of importance only under special conditions of temperatures and loadings.
   c. Compressive tests are fundamentally shear tests.
   d. For purposes of fundamental analytic research, it is necessary to use tests in which the resisting factors can be separated from one another and mathematically represented.
   e. The Hubbard-Field test cannot be mathematically analyzed, hence should be classed as an empirical rather than an analytic test.
   f. The standard compressive test also cannot be analyzed except by an elaborate combination of conditions.
   g. The use of the stabilometer with the Mohr circle method gives analytical results which moreover can be correlated with compressive tests.
3. **The Use of the Stabilometer with Mohr Circle Method**
   a. For each circle the vertical and horizontal pressures, and the speed, must be in equilibrium.
   b. Temperatures and speeds must remain constant throughout the test.
   c. The effect of the range of pressures used is a factor which must be taken into consideration.
   d. Within certain limits the effect of height-diameter ratio is the production of distortions in the Mohr diagram of such nature that mathematical correlations have not been made.
4. **The Relationship Between Dry Aggregates and Bituminous Mixes**
   a. Experience to date indicates that there is a relation between the stabilities of aggregates and mixes, but that other factors enter to such an extent that quantitative relations of dependable nature cannot yet be reported.

5. **The Problems of Practical Correlations to be Solved**
    a. The problem of transferring the results from the large height-diameter ratios of specimens which is necessary in order to avoid distortion of the resistance factors in tests, to the condition of wide tread tires on thin road pavements, and especially with relation to the qualities of the subgrade.
    b. The selection of the testing temperatures which will give the closest correlation with observed effects of flow in pavements.
    c. The problem of relating quasi-static tests to the different conditions under traffic, which introduce very short durations of pressure and also impact effects.

CONCLUSIONS

1. The Mohr circle method of using the pressure cell or stabilometer promises the most fundamental method of analysis of the various factors giving stability to a bituminous mixture, but is still a method of fundamental research not yet adapted for routine testing.
2. The correlations already discovered give promise of analyzing other tests whose meaning has been somewhat obscure.
3. There is such a wide field yet to be covered between semi-static test methods and conditions of practice that a very open mind should be maintained as to the practical value of all present tests.

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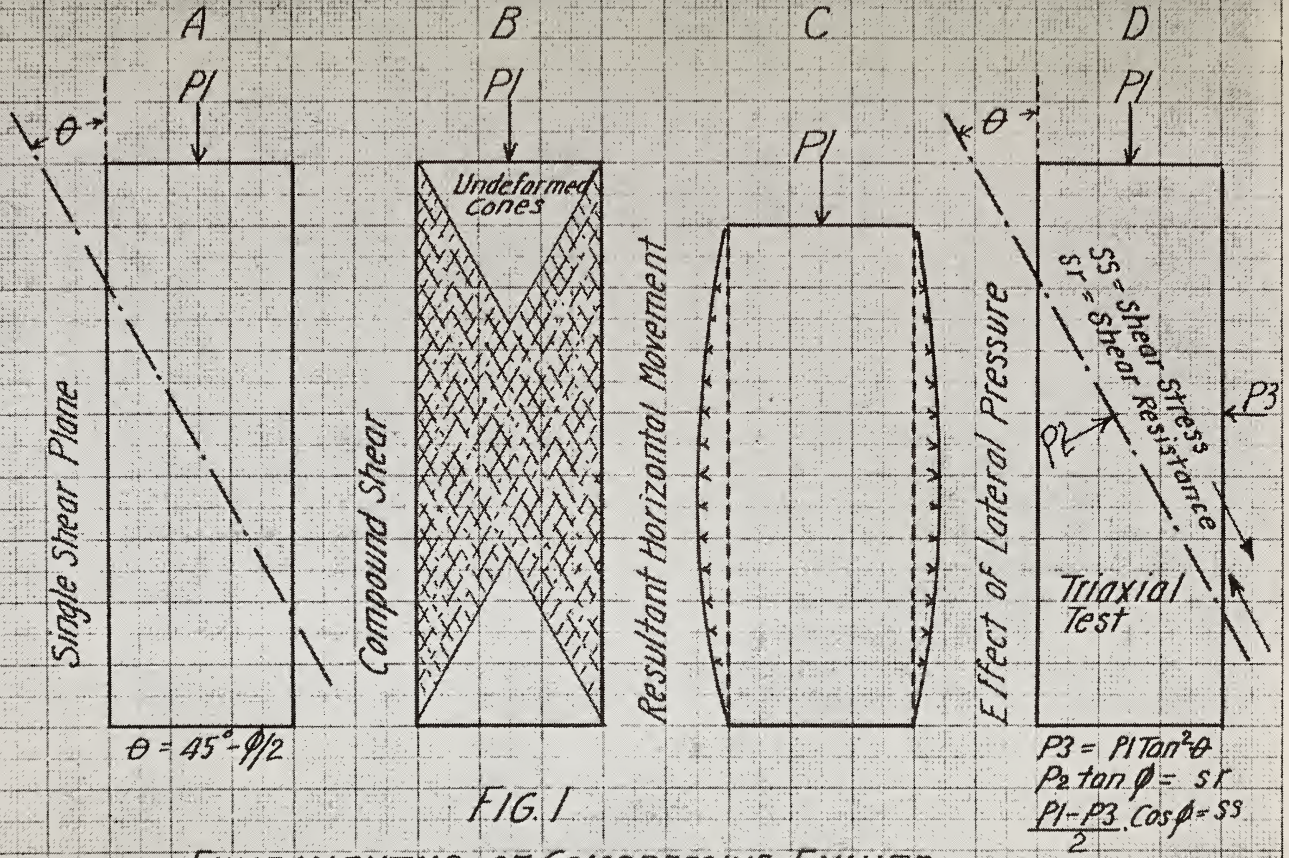
A
B
C
D
P1
P1
P1
P1
θ
θ
Single Shear Plane
$\theta = 45^\circ - \phi/2$
Compound Shear
Undeformed Cones
Resultant Horizontal Movement
Effect of Lateral Pressure
SS = Shear Stress
sr = Shear Resistance
P2
P3
Triaxial Test
$P3 = P1 \tan^2\theta$
$P_2 \tan\phi = sr$
$\frac{P1 - P3}{2} . \cos\phi = ss$

FIG. I

FUNDAMENTALS OF COMPRESSIVE FAILURE

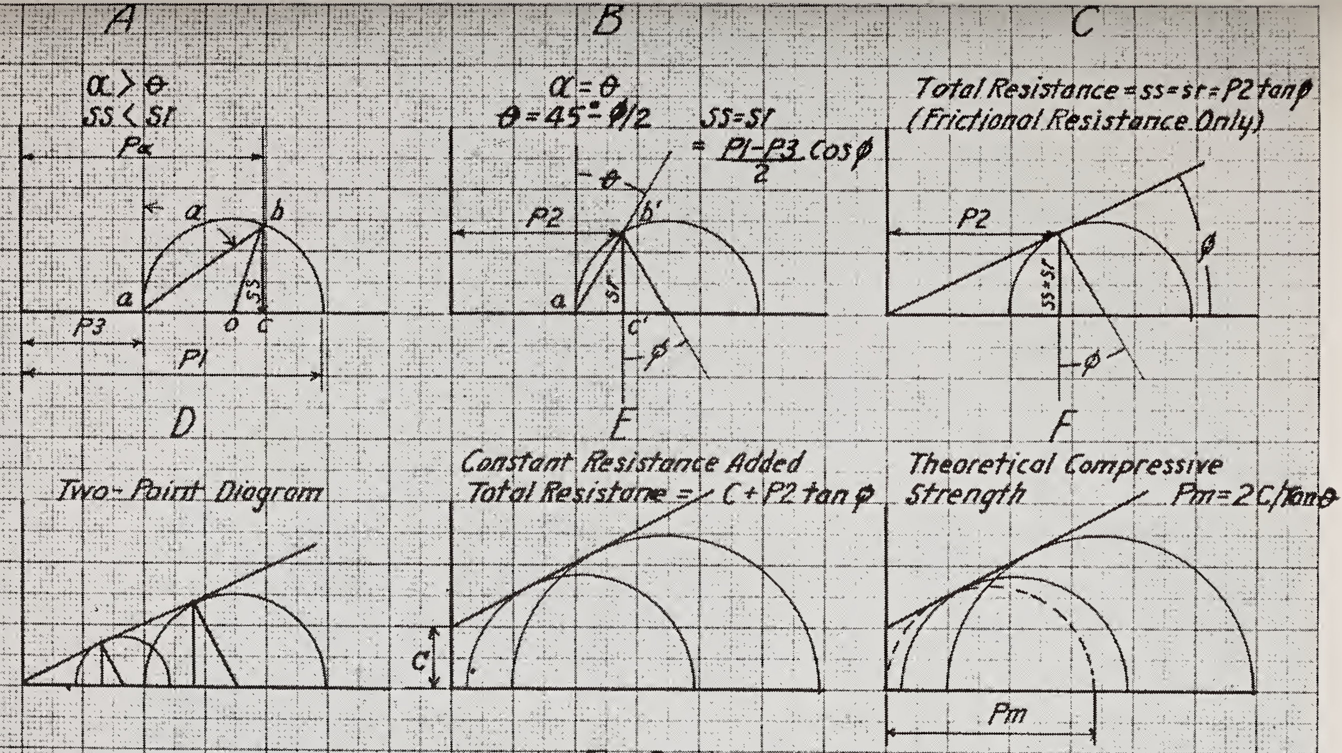
A
α > θ
SS < Sr
Pα
α
a
b
SS
o
c
P3
P1
B
α = θ
θ = 45° − φ/2
SS = Sr
= (P1 − P3)/2 Cos φ
θ
P2
b′
a
Sr
c′
φ
C
Total Resistance = ss = sr = P2 tan φ
(Frictional Resistance Only)
P2
φ
SS = Sr
φ
D
Two-Point Diagram
E
Constant Resistance Added
Total Resistance = C + P2 tan φ
C
F
Theoretical Compressive
Strength
Pm = 2C/tan θ
Pm

FIG. 2

ELEMENTS OF THE MOHR DIAGRAM

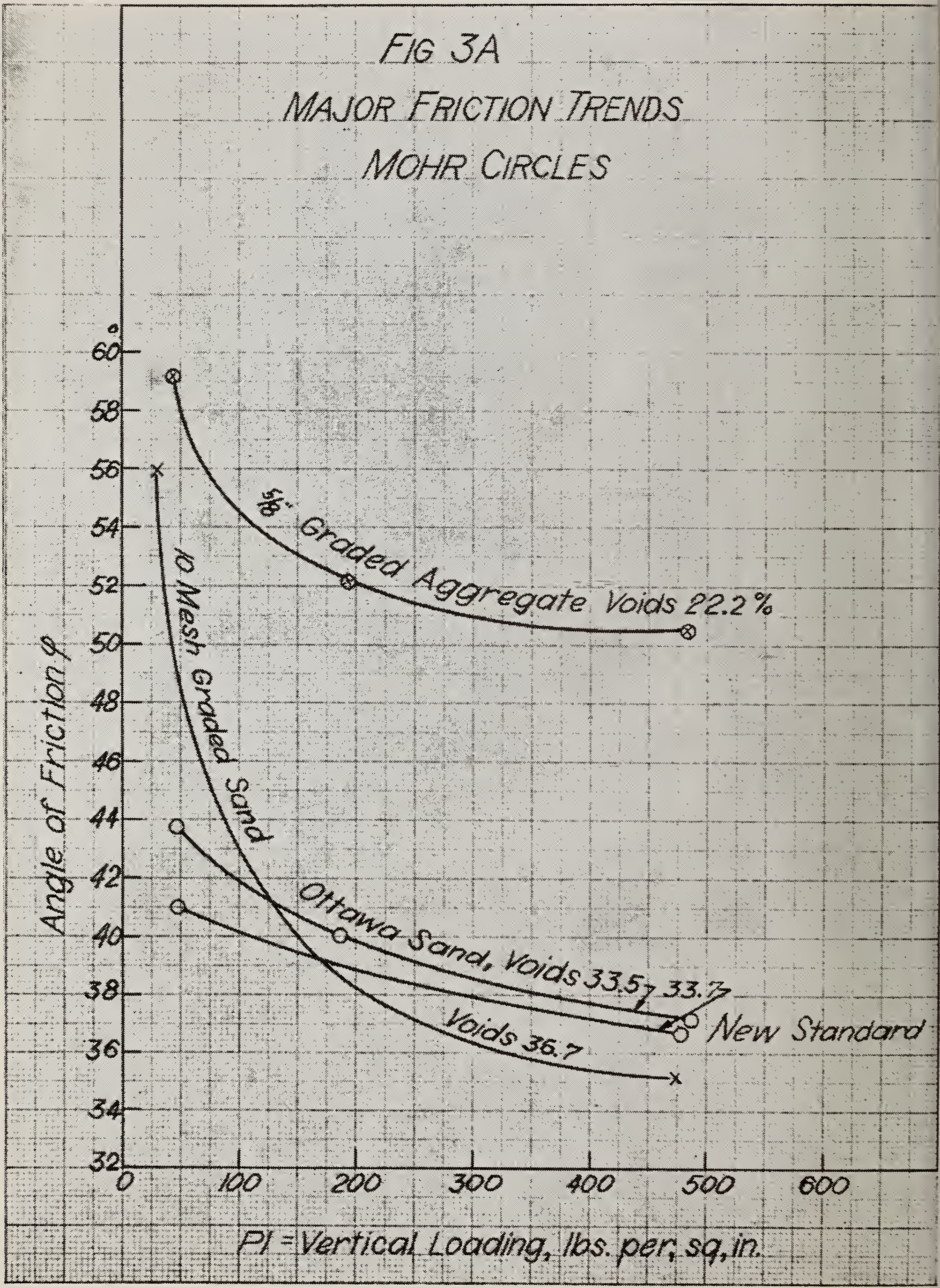

FIG 3A

MAJOR FRICTION TRENDS

MOHR CIRCLES

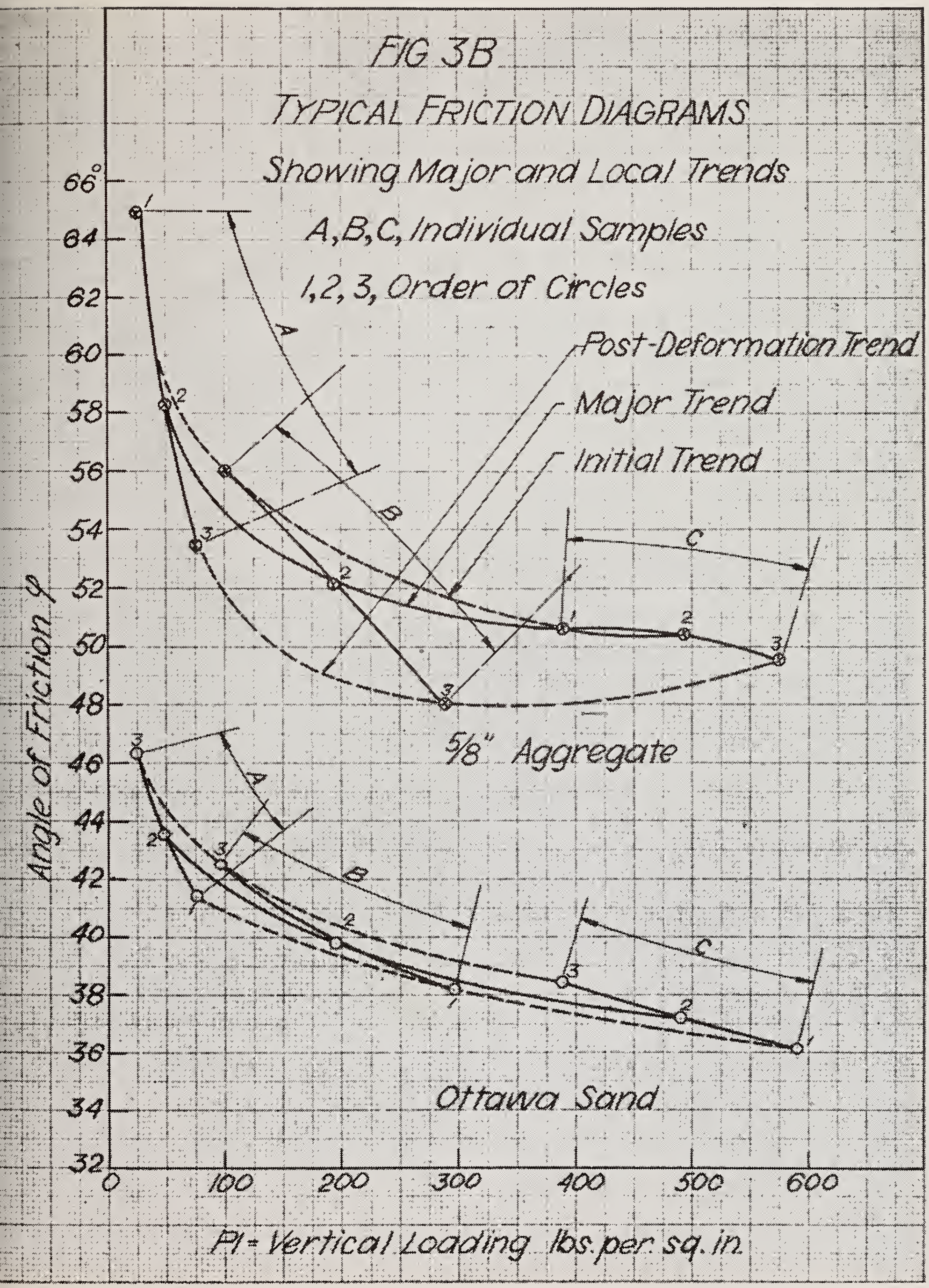

FIG 3B

TYPICAL FRICTION DIAGRAMS

Showing Major and Local Trends

A, B, C, Individual Samples

1, 2, 3, Order of Circles

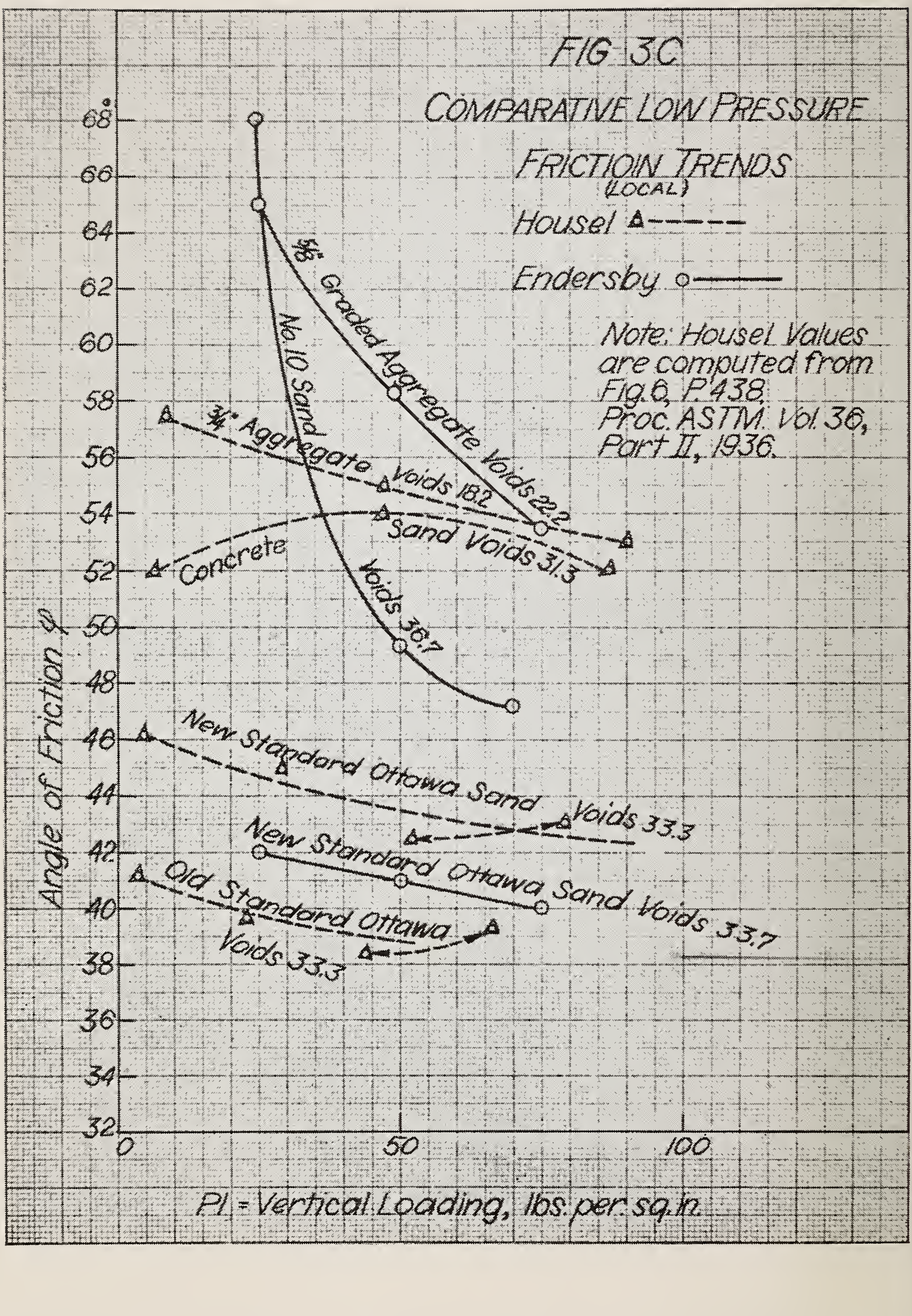

FIG 3C

COMPARATIVE LOW PRESSURE FRICTIOIN TRENDS (LOCAL)

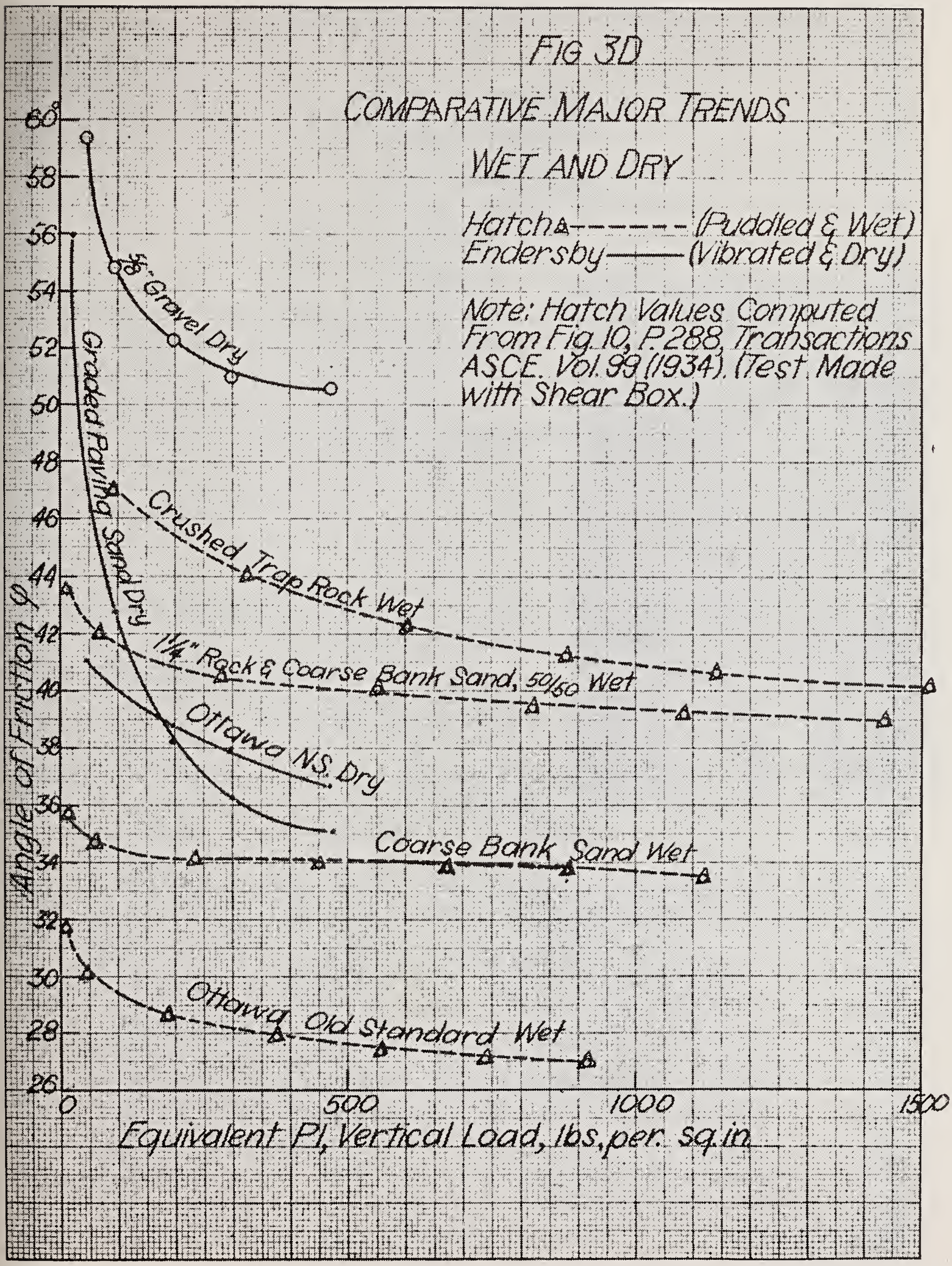

FIG 3D

COMPARATIVE MAJOR TRENDS

WET AND DRY

Hatch △ - - - - - (Puddled & Wet)
Endersby ——— (Vibrated & Dry)

Note: Hatch Values Computed From Fig. 10, P. 288, Transactions ASCE. Vol. 99 (1934). (Test Made with Shear Box.)

Correlation of Laboratory Tests of Bituminous Mixtures With Service Behavior

By

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During the last few years it has been my pleasure to have conducted a research program for the Barber Asphalt Corporation to determine which of several laboratory tests give the most reliable information regarding the performance of sheet asphalt pavements. The data obtained have proved enlightening. While the complete data have been published in two parts in the December, 1937 and January, 1939 proceedings of the Association of Asphalt Paving Technologists, the present paper is the first attempt at a complete correlation of all the physical characteristics studied and discussing their inter-dependence. In the writing of this paper it is my purpose to establish a proper perspective so that the significance of each characteristic may be fairly interpreted.

Samples were obtained after a survey of pavements in eighteen representative cities in the Eastern half of the United States. The samples were taken from pavements at least two years old and representing three typical behaviors in service, i. e., (1) the cracking type of surface which is designated as Group A, (2) the sound type which has neither cracked nor shoved, designated as Group B, and (3) the shoving type designated as Group C. This classification of each pavement was made in the field by inspection of the surfaces and examination of the service records. Information regarding the type, source and consistency of the asphalt cement, gradation and source of the aggregates, box weights and other items of identification were also obtained for each sample.

Because the itemized data is complete and generally available in the two proceedings of the Association of Asphalt Paving Technologists already referred to, we are not repeating these tabulations. Our paper includes only the tabulation of average values and the standard deviation of the data obtained in each representative group of samples classed as A, B, and C, respectively.

The data pertaining to mixtures made with Native Trinidad Lake Asphalt are not included in the presentation of this paper. Previous analyses have shown that a line of distinction must be drawn between the characteristics of mixtures containing Native Lake Asphalt and those containing asphalt derived from petroleum. A discussion of these differences may be found in the January, 1939 proceedings of the Association of Asphalt Paving Technologists. While they are significant and generally interesting they would tend to confuse the more precise discussion which is desired at this time.

MIXTURE COMPOSITION

In any correlation it is necessary to recognize the variations that may exist in the mixture composition and properties of the several ingredients. Previous analysis of the data has shown that the gradation of the aggregate obtained after the extraction of the bitumen from each sample averages practically the same for the pavements represented in each group. It appears that all of the mixtures were made with aggregates conforming to the usually accepted specifications for grading. Considering averages, as it is proposed to do in this paper, the aggregate can be considered uniform in all three groups of pavements.

The mixture composition averaged for each group of pavements is shown in Table 1.

Table 1.
AVERAGE MIXTURE COMPOSITION

Mixture component	Filler, % by wt. (–200 mesh)	Bitumen, % by weight
Group A—Average	13.5	10.4
Standard deviation	2.0	0.8
Range by std. dev.	11.5 to 15.5	9.6 to 11.6
Group B—Average	14.3	10.2
Standard deviation	2.2	0.5
Range by std. dev.	12.1 to 16.5	9.7 to 10.7
Group C.—Average	17.8	11.2
Standard deviation	1.5	0.6
Range by std. dev.	16.3 to 19.3	10.6 to 11.8

A definite trend is evident from the normal filler and bitumen content in Group B to high filler and bitumen content in Group C. The difference noted in the average values above is usually sufficient to account for the respective behavior of mixtures in these two groups. According to these data it may be said that Group C pavements are, for the most part, the result of improper design of mixture proportions or due to faulty plant control allowing the mixtures to become overfilled and excessively rich in bitumen.

Of greater importance, however, with regard to the current problems in asphalt construction and design is the fact that the average mixture compositions of Group A and Group B pavements are practically identical.

Interpretation of the data is fairly obvious; although Group C pavements may be the result of improper mixture proportions, Group A pavements must be the result of something else.

In Tables 2, 3 and 4 are shown the averaged characteristics of bitumen recovered from the samples of each group of pavements. As one might readily surmise, the hardest bitumen was found in samples of Group A and the softest in samples of Group C.

Table 2.

PENETRATION DATA ON ORIGINAL AND RECOVERED BITUMEN.

(Penetration, 100 gms., 5 sec., @ 77 °F.)

Type of Pavement	Group A	Group B	Group C
Average original penetration	49	51	55
Average penetration, recovered bit.	18	29	39
Standard deviation—Pen. recov. bit.	7	10	5
Range of pen. recov. bit by std. dev.	11 - 25	18 - 39	34 - 44
Average percent drop in penetration	64	44	29
Standard deviation of percent drop	14	21	9
Range of percent drop by std. dev.	50 - 78	23 - 65	20 - 38

In Table 2 it will be observed that the original asphalt cement in groups A and B had a nominal average penetration of 50. The shoving Group C pavements had an average penetration of only five points higher. It may be assumed that all of the asphalts used were of the usual paving cement grade and had nominal 50 and 55 penetration. The average penetration of bitumen recovered from the pavement samples however, shows a decided trend toward low penetration in the cracking type, Group A pavements with successively higher penetration in the satisfactory and shoving, Groups B and C types.

The range of penetration in the recovered bitumen indicative of cracking and non-cracking pavements as shown in the table is in fair agreement with the range reported by Hubbard. As we shall see, however, the range covered in the present data is somewhat more tolerant. Thus, according to the data the following limits would be reasonable:

> With penetration of the recovered bitumen, less than 18, the pavements are definitely of the cracking type.
>
> When the penetration of the recovered bitumen is between 18 and 25, the pavements are prone to crack.
>
> When the penetration of the recovered bitumen is over 25, the pavements are generally free from cracking.

To eliminate the effect of slightly different original penetrations, the second half of the table was constructed to show the results obtained from the data on a percentage drop in penetration basis. On this basis the data shows:

> Cracking pavements — recovered bitumen penetration drop greater than 65%.

Questionable pavements — penetration drop between 50 and 65%.
Non-cracking pavements — penetration drop less than 50%.

According to these limits, specifications of the type promulgated by the states of Michigan and Virginia, stipulating that the penetration of bitumen recovered from a mixture shall not be less than 50 per cent of the original penetration, are justified.

The averaged data for ductility of the bitumen recovered from the pavement samples is given in Table 3. There is no comparison with the original ductility because no records were generally available. It can only be assumed that the original ductility of the asphalt cements was over 150 cms. in most samples.

Table 3.

DUCTILITY OF BITUMEN RECOVERED FROM PAVEMENT SAMPLES

(Ductility cms., 5 cms. per min. @ 77° F.)

Type of Pavement	Group A	Group B	Group C
Average ductility, recovered bit.	6	40	70
Standard deviation	18	36	28
Range by std. dev.	0 - 24	4 - 76	42 - 98

Interpreting these data in the same manner as the penetration data in Table 2 will indicate the following limits:

When the ductility is less than 4 cms., the pavements are definitely of the cracking type.

When the ductility is between 4 and 24 cms., the pavements are prone to crack.

When the ductility is over 24 cms. the pavements are generally free from cracking.

The averaged data for Ring and Ball melting points are given in Table 4 followed by similar interpretation.

Table 4.

RING AND BALL MELTING POINT OF THE BITUMEN RECOVERED FROM PAVEMENT SAMPLES.

(Melting point, R&B, deg. F.)

Type of Pavement	Group A	Group B	Group C
Average melting point	178	148	142
Standard deviation	18	12	4
Range by. std. dev.	160 - 196	136 - 160	138 - 146

These data may be interpreted as follows:

When the melting point of the recovered bitumen is over 160 deg. F., the pavements are generally prone to crack.

When the melting point is less than 160 deg. F., the pavements are generally of the non-cracking type.

When the melting point of the recovered bitumen is between 136 and 146 deg. F., the pavements are tending toward the shoving type.

When the melting point is less than 136 shoving would be almost certain in hot weather.

The melting point data for Groups B and C are of particular interest with regard to the mechanism of service behavior of mixtures. The average melting points of Groups B and C are 148 and 142 degrees F. respectively. Considering the fact that attainable summer temperatures in asphalt pavements is generally considered to be about 140 degrees F., suggests the possibility that resistance to cracking is at least partly enhanced by the fact that the asphalt cement in these mixtures may be soft enough periodically to allow the asphalt cement to flow or migrate sufficiently so that the mixtures can be in some measure self-healing.

Assuming that the asphalts used in the manufacture of the pavements sampled were practically alike in their original characteristics, the changes observed in Group A are indicative of oxidation as the primary cause of hardening. The penetration and ductility are low and the melting point is high for the bitumen recovered from Group A samples all of which is typical of blown asphalt. There is some discussion at the present time suggesting the use of softer asphalt cements in mixtures so that softer bitumen would be found in the finished pavement. The advisability of using this expedient may be questioned since it is the blown characteristics of the bitumen that are more undesirable than the actual hardness, or high viscosity, of asphalt cement. If the characteristics which appear to be so undesirable are the result of oxidation, it would be more proper to concentrate our efforts on elimination of its cause from the processes of mixture manufacture and pavement construction. This point should be carefully considered since it must be remembered that in general the softer the asphalt, the more easily it is oxidized.

Age of Pavements

According to facts in the foregoing data it is apparent that the cracking or non-cracking of a pavement depends primarily upon the extent to which the asphalt cement is hardened or oxidized in the paving mixture. There is a question as to whether the asphalt cement has been hardened to its present condition during the mixing operation prior to laying or whether some of the hardening is the result of aging of the asphalt in the pavement. A study of the age of pavements represented in each group may be helpful in considering this question.

Table 5.

AGE OF PAVEMENTS WHEN SAMPLED

Type of Pavement	Group A	Group B	Group C
Average age in years	10	9	14
Standard deviation	5	7	6
Range of age by std. dev.	5 to 15 yrs.	2 to 16 yrs.	8 to 20 yrs.

In Table 5 it will be noted that the soft, shoving type, Group C has the greatest average age, 14 years. Groups A and B are approximately alike, averaging 10 and 9 years respectively. If age were much of a factor in the hardening of the asphalt we should certainly expect to find a much greater difference between the ages of Group A and Group B to account for the considerable difference in the characteristics of the bitumen recovered from these two groups. Unless the bitumen in Group A has hardened for some unknown reason much more rapidly than that in Groups B and C, the hardening of the asphalt is accomplished most probably during the mixing and laying operations at the outset of the life of the pavement. It is most probable therefore that the characteristics of a pavement will change but little during the course of ten or fifteen years as covered by the data. On this premise, tests to determine the characteristics of a freshly laid asphalt pavement will be indicative of the future tendency of the pavement to crack.

PHYSICAL CHARACTERISTICS OF MIXTURES

Up to the present point in this paper we have discussed properties pertaining to the mixture composition and characteristics of the various ingredients of mixtures. It has been shown that the primary influence on service behavior of cracking and non-cracking pavements resides in the characteristics of the asphalt cement as it exists in the finished mixture. Shoving pavements result generally from excess bitumen and overfilled mixtures of faulty design. A properly proportioned mixture containing a suitable bitumen when laid, will accordingly result in a satisfactory pavement.

Tests that are easy to run have been devised and used to facilitate proper design of mixture proportions insuring adequate stability to avoid the possibility of constructing pavements which will shove like the Group C type. Tests are also available to measure the physical characteristics of the asphalt cement and bitumen recovered from mixtures. The recovery process is a rather lengthy and exact procedure, however, to recover the necessary bitumen for a sample to test.

Since tests for the physical characteristics of the whole mixture are generally speedy and simple to run it is felt they would be very useful for the design and control of mixtures, particularly if they could distinguish between satisfactory and unsatisfactory characteristics in a mixture resulting directly from the influence of characteristics of the bitumen. The use of such tests capable of diagnosing correctly the necessary characteristics of a mixture would be a large step in the direction of simplification by reducing the number of tests now considered to be essential for control and which are run on the ingredients of the mix. Asphalt construction is about the only branch of engineering in which the quality and application of a composite material are judged on the basis of tests evaluating characteristics of the component parts.

The correlations which follow have been made with the specific purpose in mind of finding out how much we may expect along these lines from a few of the currently advocated tests.

Density and Voids of Pavement Samples

The density and voids of mixtures were determined both on the pavement as received and on standard 2″ dia. x 1″ tall briquets. The standard briquets were made from samples of the paving mixture, heated under conditions which are known to minimize hardening or modification of the asphalt cement and compressed in a double plunger mold at 325° F. under a load of 5,000 pounds per square inch applied for ten seconds and immediately released. The voids were determined according to the usual formulae using the density of the pavement and briquets and the densities of the bitumen and aggregates obtained from the bitumen recovery and extraction procedures.

Table 6.

DENSITY AND VOIDS IN THE MIX

	DENSITY					
	Standard Briquets			Pavement Samples		
	Group A	Group B	Group C	Group A	Group B	Group C
Average density	2.21	2.20	2.21	2.15	2.20	2.21
Standard deviation	.04	.04	.03	.08	.05	.03
Range by std. dev.	2.17-2.25	2.16-2.24	2.18-2.24	2.07-2.23	2.15-2.25	2.18-2.24

	VOIDS IN THE MIX					
	Standard Briquets			Pavement Samples		
	Group A	Group B	Group C	Group A	Group B	Group C
Average voids in mix	4.3	4.2	3.0	7.1	4.6	4.0
Standard deviation	0.9	1.4	0.8	3.4	2.0	0.4
Range by std. dev.	3.4-5.2	2.8-5.6	2.2-3.8	3.7-10.5	2.6-6.6	3.6-4.4

The data in Table 6 show the density of the standard briquets of all three groups to be practically the same. The voids in the standard briquets of all three groups show that the aggregates must have been compacted to practically the same amount since the voids in the mix is affected only by the amount of bitumen these mixtures are known to contain. Comparing the pavement samples with the standard briquets, it will be seen that while Group B and Group C densities are alike the Group A pavement samples are considerably less dense than the standard briquets. The voids in the Group A pavements are correspondingly higher than the voids in the standard briquets.

It will be remembered that the mixture composition and aggregate gradation of Groups A and B are practically identical. The method and temperature of rolling these cracked type of pavements were undoubtedly the same on the average as those used for the satisfactory ones. The average traffic conditions were also probably very similar. It follows then that the lack of density in the pavement must be due to the hard asphalt found in this type of pavement.

One function of the asphalt cement or bitumen in a paving mixture is to lubricate the mixture and facilitate compression in the pavement to the extent that maximum density and minimum voids may result. The lubricating value of the bitumen depends both upon the consistency and thickness of films on the aggregate. According to these considerations it is very possible that the lubrication of pavements in Groups B and C is entirely adequate for compaction to the standard density in the pavement. By the same premise it follows that the hard asphalt contained in Group A pavements is not a good lubricant and, therefore, must have interfered with good compaction.

Since it is our belief that the characteristics of the bitumen in these mixtures is but little different than it was at the time of laying the pavement, a corollary would be that the interference to compaction existed then as now and that the lack of density in the Group A pavements was evident at the time of laying the mixture and could have been detected if the pavement density had been compared with the density of a standard briquet. It seems advisable, therefore, to make this comparison on all new construction since by this simple test it is possible to detect evidence of serious hardening of the asphalt without resorting to actual measurement of the characteristics of the bitumen recovered from the pavement.

Mechanical Test Characteristics

The physical characteristics of mixtures as measured by mechanical tests in the laboratory are listed in Table 7 with the averaged data for pavements in the three groups.

Table 7.

PHYSICAL CHARACTERISTICS OF PAVING MIXTURES BY VARIOUS MECHANICAL TESTS.

Type of Pavement	Group A	Group B	Group C
Hubbard-Field Stability, pounds	3975	2400	1045
Standard deviation	1520	1020	450
Range by std. dev.	2455-5495	1380-3420	595-1495
Skidmore Shear, pounds/sq. in.	79	39	21
Standard deviation	37	13	14
Range by std. dev.	42-116	26-52	7-35
Compressive strength, lbs./sq. in	1285	935	520
Standard deviation	415	265	220
Range by std. dev.	840-1670	670-1200	300-740
Elastic limit, pounds/sq. inch	1070	785	420
Standard deviation	345	205	175
Range by std. dev.	725-1415	580-990	245-595
Modulus of Elasticity, lbs./sq. in.	18300	13000	6600
Standard deviation	8000	3900	2200
Range by std. dev.	10300-26300	9100-16900	4400-8800
Modulus of Permanent Deformation, lbs./sq. in	6300	3600	1300
Standard deviation	2600	1600	900
Range by std. dev.	3700-8900	2000-5200	400-2200

The physical characteristics averaged in Table 7 were determined by testing standard 2″ dia. x 1″ tall briquets according to the test procedures recommended by the authors of each test. The stability and shear tests are conducted at 140 degrees F. The entire testing machines with the exception of the hand wheels and indicating dials are, in the Barber Asphalt Corp. laboratory, enclosed in an oven maintained at that temperature.

The compression test is conducted at 77 degrees F. either at room temperature when available in air, or in a water bath. A Baldwin-Southwark hydraulic compression testing machine was used. It is operated so as to apply a constant rate of strain equal to .05 inch per minute. The four physical characteristics: Compressive strength, elastic limit, modulus of elasticity, and modulus of permanent deformation are all determined from an autographic load/deformation curve obtained during the testing of each briquet.

The compressive strength is the maximum load recorded in the testing of a briquet and converted to pounds per square inch. The elastic limit is determined as the point of deviation of the load/deformation curve from the straight line portion registering a constant ratio of stress to strain; the straight line portion of the curve is used in the usual manner to calculate the modulus of elasticity. The modulus of permanent deformation is calculated from the downward sloping straight line portion of the load/deformation curve obtained after the compressive strength has been exceeded. The sign is ignored in the data. In order to judge the significance of the data the modulus of permanent deformation may be thought of as a "break-down" modulus indicating the rapidity with which the bond strength of the mixture is lost if the mixture is overloaded. A small value of the modulus indicates a material showing great reluctance to give or flow when overloaded.

Correlation of Physical Tests with Service Behaviour

Referring to Table 7 it is apparent that by any of the tests studied, Group A mixtures show the greatest factor of resistance and Group C mixtures the least. Group B mixtures average intermediately. This is apparently in direct relationship to the characteristics of the bitumen recovered from the pavements. All other factors being equal, as they are in the set of samples in the present correlation, the characteristics of the bitumen have a great influence upon the characteristics of the pavement mixture.

Considering the range of characteristics in each of the three groups it will be observed there is quite an overlap of the data between one group and the next. The extent of this overlap is influenced by the relative ability of each test to properly identify the classification of the largest number of samples. The data for such consideration are given in Table 8.

According to Table 8 there were 19 samples of Group A pavements, 23 samples of Group B pavements and 9 samples of Group C on which complete data are available. Using the Group B range of each characteristic from Table 7, the itemized data in the previously published papers was inspected to find how many samples in each group were properly identified by these limits with respect to the group in which they were originally classified when sampled from the street. Thus, a sample originally classed as Group A and having a stability greater than 3420 lbs. was considered correctly identified. If a sample in the same group had a stability less than 3420 lbs. it was conisidered incorrectly identified. In the same manner a sample classed in Group B was considered correctly identified when the value of its characteristic fell within the range of Group B and incorrectly identified if its value fell without the range. In like manner correctly identified Group C samples have characteristic values less than the lower limit of the Group B range, etc. The number of samples correctly identified is shown in the left hand column under each group in Table 8. The right hand column shows approximate percentages of the samples identified correctly by each test. The parentheses are used to indicate which characteristics appear to have the best ability to make proper identification in each group, i. e., show the highest percentage of samples correctly identified.

Table 8.

SAMPLES OF EACH GROUP OF PAVEMENTS CORRECTLY IDENTIFIED THE GROUP B RANGE OF STANDARD DEVIATION FOR EACH OF THE PHYSICAL CHARACTERISTICS

Type of pavement	Group A		Group B		Group C	
	19 Samples		23 Samples		9 Samples	
	No.	%	No.	%	No.	%
Stability	11	60	15	65	7	(80)
Shear	14	(75)	14	60	6	65
Compressive strength	12	65	15	65	7	(80)
Elastic limit	11	60	15	65	8	(90)
Modulus of Elasticity	10	55	17	(75)	6	65
Mod. of Perm. Deformation	15	(80)	19	(85)	7	(80)

Note: Parentheses indicate tests correlating greatest number samples.

The data given indicate that shear and modulus of permanent deformation identify the Group A samples better than any other characteristic. The modulus of elasticity and modulus of permanent deformation excel in the identification of Group B samples; and, any characteristic with the exception of shear and modulus of elasticity satisfactorily identifies Group C samples. From the point of view of this correlation certain qualities associated with different kinds of service behaviour may be discussed.

The hard, cracking, Group A type of pavements definitely reflect the nature of the bond furnished by the hard, oxidized in quality, bitumen through their high resistance to shear and lack of plastic property when overloaded. These characteristics are the most pronounced in Group A pavements and show their predominance by the nature of tests that give the best correlation. Thus too in the samples of the other two groups. Group B samples are best identified by the modulus of elasticity and the modulus of permanent deformation. This means simply that the pavements of this type have a certain amount of necessary resilience under ordinary loads and tend to "give" reluctantly under over-loads. The pavements of this group seem to have the quality or ability to adjust themselves under loads to the extent that large stress is distributed and relieved without actual rupture or cracking of the pavement. These qualities are of course related to the softer consistency of the bitumen found in Group B pavements, i. e., a suitable penetration, ductility and melting point. Compressive strength elastic limit and stability are suitable primarily in the identification of Group C mixtures. These characteristics are fundamentally measures of load supporting strength and serve only to indicate what is a minimum value tolerable in stable mixtures.

As a result of this correlation it is safe to conclude that no one characteristic may be considered as a suitable standard for the complete identification of a mixture with regard to its potential service behaviour. While it is possible through use of upper and lower limits to apply only one test for the identification of mixture behaviour, the accuracy of such a test for the purpose would be only two or three to one at best. In order to more accurately identify the possibly behaviour of a mixture on the basis of laboratory tests of the whole mixture, it will be necessary to consider the use of at least two tests ordinarily, or at least the measurement of more than one physical characteristic.

Thus the stability test might be used in conjunction with the shear test. Giving due consideration to the fact that the shear test is more accurate than the stability test to identify Group A mixtures and that the stability test is better to identify Group C, the limits of Group B given in Table 7 may be used as criteria for the following specification for suitable characteristics of a mixture:

Stability, Hubbard-Field, lb. @ 140 degrees F.	1380 to 3420
Shear, Skidmore, lbs./sq. in. @ 140 degrees F.	26 to 52

If the stability is found to be below 1380 lbs. the mixture will most probably be of the type found in Group C. The stability and shear strength of the most satisfactory mixtures will be within the above ranges. If the shear strength exceeds 52 lbs. per sq. in. the mixture will very probably be of the type found in Group A even though the stability may be less than 3420 lbs.

The compression test with its autographic record of the load/deformation curve furnishes in one test all of the data necessary for a somewhat better identification of mixtures in any of the three groups. The significant characteristics, compressive strength, modulus of elasticity and modulus of permanent deformation are all available from the one test. The limits from Table 7 which may be used as criteria for the specification of mixture characteristics follow:

Compressive strength, lbs./sq. in., .05″/min. @ 77 degrees F.	670 to 1200
Mod. of elasticity, lbs./sq. in., .05″/min. @ 77 degrees F.	9100 to 16900
Mod. of Perm. Deform., lbs./sq. in., .05″/min. @ 77 degrees F.	2000 to 5200

If the compressive strength is found to be less than 670 lbs./sq. in. the mixture will most probably be of the type found in Group C. If the modulus of elasticity lies between 9100 and 16900 lbs./sq. in. the mixture will perform satisfactorily provided that the compressive strength is over 670 pounds/sq. in. and the modulus of permanent deformation is less than 5200 pounds/sq. in. If the modulus of permanent deformation exceeds 5200 pounds/sq. in. the mixture will most probably perform like the type found in Group C.

While it is not our purpose to advocate one test in preference to another the advantages of the compression test should not be overlooked. In a single test three essential mixture characteristics are measured. These will make it possible to interpret the possible behaviour of a mixture in service with greater success than either the stability test or shear test operated singly or with their data interpreted jointly. Even if the compression test is used by itself merely to measure the one characteristic, compressive strength, its accuracy is slightly better than that of the other tests. It needs hardly be pointed out further that the compression test does not require the use of any special machine or testing equipment other than that generally found in almost any material testing laboratory.

In further consideration of testing or specification for control purposes, the relative density of the pavement and a standard briquet of the same mixture may be generally useful in practice to detect oxidation of the asphalt cement in the mixture during the mixing and laying operations. When a carefully and thoroughly rolled mixture does not compact to at least the same density as a standard briquet of the same mixture, the asphalt has most probably been damaged, very often seriously.

Supplying Vitamins and Eliminating Bugs in Asphaltic Road Material Specifications

By

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Even though bituminous substances have been used for a variety of purposes for hundreds of years it has only been during the past fifteen or twenty years that asphaltic material treated highway surfaces emerged from whatever status they were in, and became a serious business of large and rapidly growing proportions. This emergence sharply increased highway surface treatment activities and the practice of "oil treating" highway surfaces soon spread far and wide. This rapid growth gave rise to problems only research can solve.

Ever since asphaltic treatment of highway surfaces has so extensively occupied the attention of engineers the 'Gordian Knot'' has been to develop specifications which would insure a good, live, durable weather resisting product and at the same time, with accuracy and finality screen out materials below a feasible and economically sound standard of excellence for the purpose to be served. In the beginning specifications were the best that could be expected in the absence of much information. Naturally they were full of imperfections, woefully lacking in vitamins, burdened with excess baggage and in some cases the "thou shalts and/or thou shalt nots" made it utterly impossible to produce a superior product. In addition to this there existed a chaotic condition; there was no organized cooperative effort existing to any worthwhile extent and as a result there was a multiplicity of specifications covering products desired to be used for the same purpose essentially but none of them spoke the same language.

It was a long step in the right direction when the industry and profession, through the initiative of the United States Bureau of Public Roads and the Asphalt Institute, got together, evolved a common language, a simplified scheme of analysis and classified asphaltic road materials into a reasonable number of types and grades. While this was a splendid accomplishment no effort was directed toward the production of provisions compelling a good, live, durable weather resisting product. So, good, bad and indifferent products continued to find their way into highway surfaces all over the country under any and all specifications simply because provisions aimed to secure quality were either absent or woefully anemic and wholy inadequate to cope with the situation. The solution of the problem therefore seems to lie in evolving some test, procedure or process which will dependably separate the inferior from the superior and incorporate it in the specifications. This is a lot easier said than done, but we believe material progress has been made toward the accomplishment of this end through rational approach to the problem and that indispensable activity, research.

Many have devoted much time and attention to the problem with varying degrees of success. New Mexico's efforts along this line seem attended with gratifying results. To us the logical approach seemed to consist of the three following steps:

1. Use of only crude oils which were potentially capable of producing good, live, durable, weather resisting products.
2. Refine the crudes without injury.
3. Keep impurities and deleterious substances to a feasible minimum.

These three steps describe a principle of Spartan simplicity. However, each step must be broken down into its respective technology. One must know what crudes are potentially capable of producing the desired quality of products and how to identify them before dependable selection is possible. Fortunately we are already furnished with information authored by recognized authorities, to the effect that asphaltic base crudes and semi-asphaltic base crudes (in the order named) produce the grades of asphaltic road materials most desirable. We are also informed to a very considerable extent as to the characteristics and properties of these crudes, as well as those of the inferior paraffin or non-asphaltic base crudes, and tests which serve well for identification. In view of this we feel that the process of vitamining and vitalizing the specifications may well be started by requiring that each and every crude which is proposed to be used in the refining of asphaltic road materials for or by the State Highway Department be sampled and submitted to the State's Highway Materials Testing Laboratory for examination and approval before it is used either by itself or in combination with some other crude or crudes.

A second shot of vitamins is supplied by the specifications stating that the refined product shall not be burned, cracked or otherwise injured in the refining process and backing that requirement up with ''anti-cracking guns'' that will do a good job of gunning out overheated and cracked oil without doing the same thing to very good, quite desirable uncracked material. The resistance to oxidation test and the color of asphaltenes test seem to be two very simple tests of apparently criterion value and differentiating dependability. While neither of these tests would detect a drop of cracked material in a carload, nevertheless, they both are sufficiently sensitive to limit adulteration or overheating to a very small percentage and dependable enough not to eliminate uninjured material.

A third vitamin treatment consists of deleterious and extraneous material limitation. Of all materials which may be regarded as deleterious we feel that paraffin is the worst. Not only is it indicative of inferiority inherent in the oil but because of its temperature sensibilities it renders the highway surface weakest in summer, the very time it has the heaviest traffic load to carry.

The final vitamin injection may take the form of a statement in the specifications to the effect that borderline failing material will be definitely rejected rather than allowed to become controversial on the grounds that there is no justification for permitting the use of doubtful material.

Now, having prescribed a few vitamin shots for the specifications let us turn our attention to eliminating some bugs. Looking the situation over we find that as the vitamins take effect they have a very pleasing tendency toward minimizing bugs. There still remains however a few bugs of various species which should be eliminated if possible. We may not be able to secure 100 per cent elimination but nevertheless we feel that a trip through the delousing plant would in many cases be quite beneficial.

The long drawn out time consuming test that has little or no real significance or serves no real purpose not already and probably better served by some other and far more simple test is one species we feel that the specifications could do without with benefit. Particularly so if the test in point is a prerequisite to another of like calibre. Another kind of test that may well be eliminated is any which is highly involved or thickly populated with acute exactnesses which provide many points of vulnerability and rich grounds for controversy. Tests which work against each other or try to force one material to have something which cannot be injected in to it without the possibility or certainty of injury. Again we have the test which is perfectly good for its original purpose but not applicable to what seems to be a similar purpose. And still another test whose results are characteristic of both a good product and a poor one. And finally we have a whole family of bugs which may be just exactly the thing for refinery control or as a part of research but which are completely out of place in specifications. Many tests may be masterpieces of professional accomplishment serving their original purpose and having still other adaptabilities with or without modification, yet if they do not serve a specification purpose nor can feasibly be made to do so they become just as much of a bug in the specifications through absolutely no fault of their own, as any other requirement which may be of little value or even detrimental.

Too much standardization which goes beyond its virtues and compels exactly the same inflexible prescription for all ills regardless of local conditions or special considerations is a less technical bug but nevertheless if present should be eliminated. Standardization is a most essential thing but beyond its optimum point it can and does do harm.

To summarize the above the following is given (subject to editing) as generally describing our idea of the substances of a typical vitamined-deloused asphaltic road materials specification:

1. A sample of each crude from which the refined product is proposed to be refined shall be submitted to the State's Highway Depart-

ment Laboratory for examination and approval and only approved crudes are to be used in refining asphaltic road materials for use for or by the state highway department. (For quality).

2. The finished product shall not be cracked, burned or otherwise injured. Neither shall it contain any burned, cracked or otherwise injured material. (For quality).

In addition to the above the finished product shall meet the following pecifications:

1. Water and sediment limitation (for purity).
2. Flash Point (fire hazard, type identification, etc.)
3. Consistency (selected according to type of construction).
4. Paraffin and other deleterious substance limitation (for quality and purity).
5. Anti-cracking tests (for quality).
6. Solubility in CC14 (for identification and purity).
7. Mechanical tests to identify type and grade of material.
8. Other test to serve any special purpose.

In the foregoing we have confined ourselves to liquid and solid petroleum products and perhaps have not even covered this field very well. Specifications covering the aggregates to be treated could no doubt likewise be put through a like clinic with benefit. But we did not intend to deal with other than petroleum and its asphaltic road material products. In doing so it is hoped that there may be found something which may be of some little value toward securing specifications which will accomplish the end which we all desire.

Construction of Low Cost Bituminous Surfaces

By

J. L. STACKHOUSE
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With the advent of the modern, high speed motor vehicle and a dust-conscious bump-conscious state of mind of the average motorists, the position of the majority of the County Road Engineers has become ever more difficult than in former years, by reason of the demand of the traveling public for year round, dustless, smooth road surfaces.

In one way the motor car owner cannot be blamed for his attitude, for there is not a person in this room who really likes to take your clean car out on a dusty road and come back with it dusty and dirty inside and out and yourself ready for the cleaners. A few years ago this condition was expected on any trip. Present-day conditions have changed and we are used to better driving conditions.

My purpose in coming before this meeting of the County Road Engineers is to discuss some of the methods used by the Highway Department in constructing low cost bituminous mats and if possible bring to you a few new ideas that may help with the solution of improving the surfaces of your roadways.

There have always been many varied opinions as to the best methods of constructing asphalt pavements and the designing of such surfaces. It has always been a very controversial subject. Methods of constructing low cost bituminous surfaces are subject to a greater degree of diversity of opinions and seemingly opposing ideas are being successfully employed in various parts of the United States and even in Washington.

To the layman this undoubtedly is very confusing, but when one considers that in few other industries do local materials and conditions play such an important part and that while methods used with one kind of materials are successful in one locality, the same methods may cause failure, or be too expensive, if employed in another.

By reason of the diversity of climatic and sub-base conditions and materials, the problems in various parts of the state are rarely similar in all respects, and helping to determine the correct procedure for each section is very interesting to me.

BASE OR FOUNDATION CONSTRUCTION

Before we take up the methods of constructing the bituminous surface, I would like to discuss briefly factors that contribute most of the success or failure of a bituminous surfaced roadway or as a matter of fact any pavement surface whether rigid or flexible in type. The factors I refer to are drainage and a stable foundation. These two items are inseparable and go hand in hand. The old adage of ''No chain is stronger than its weakest link'' can aptly be paraphrased to ''No roadway surface is stronger than its foundation.'' This is more noticeable or quickly apparent in the low cost thin mat bituminous surfaces than the heavier pavements, but the results are the same in the end.

You men who live in counties that are covered or underlaid with a subsoil of natural sand and gravel are fortunate and have no great foundation problems. Adequate drainage is your primary consideration in any oiling program.

The men who in their counties have clay sub-bases or heavy silts or dirts upon which to construct a satisfactory road surface have an entirely different set of conditions that are many times perplexing and difficult to solve at a cost justified by traffic using the roadway or money available.

A few years ago the Highway Department came to the conclusion that to stabilize or construct a solid foundation over clay or unstable subsoils, it was advisable to lay, when available, coarse open graded ballast mats of crushed rock of nine inches (9″) to sixteen inches (16″) in thickness and the problem would be whipped. But as the next few years rolled around we still continued to have soft areas develop that caused failure of the bituminous mat in the immediate vicinity of the break on these highways. In a few cases the soft areas were removed to a depth of twenty-four to thirty inches and backfilled with the same ballast rock, with no better success.

The conclusion as to what was happening is that during the winter months and during wet weather, water was being carried up under the ballast section by capillary action. Since the surface of the roadway was sealed against evaporation of the water by the oil mat, the water continued to collect until a very unstable condition existed in the clay subsoil. When loads traveled over the highway with accompanying vibration and wheel pressure, the saturated, slippery clay was squeezed up into the open areas or voids of the ballast layer. Being lubricated by the clay, the rock pieces were pressed down into the clay with each passing vehicle until we had a complete interchange in position between the ballast and clay mass. The rock was lost.

In order to correct this situation, a sand or ¾″ — 0 surfacing material varying from one (1) to six (6) inches in depth, according to the need, is now being placed on the subbase ahead of the foundation courses with remarkable success. During wet weather when there is an excess of moisture, the unstable clay will start to come to the surface but is blanketed off by the sand cushion and the sand will be pushed into the voids in the ballast. Sand has no lubricating value.

In addition, if the sand or cushion layer is of sufficient thickness, as excess or free water develops under the ballast layer, it will be carried to the sides through the cushion and extreme unstable conditions cannot occur under the bituminous mat. It is necessary that the sand cushion as well as the ballast and base courses be placed from shoulder to shoulder of the roadway section. Wherever possible sandy gravel selected roadway borrow is used instead of the ballast and sand cushion courses and acts as a sand cushion as well as a stable foundation course. It is much cheaper to construct in most cases.

Before starting an oiling program it is mandatory upon every engineer to be thoroughly satisfied the road surfaces he intends to treat have sufficient foundation. It may be necessary to prepare a construction program extending over a period of years, first preparing a foundation with adequate drainage and oiling the second or third year as money is available.

If the subbase is not composed of sand-gravel or granular material, soil tests should be made on the material to find the soil constants to determine whether or not it will provide suitable foundation. Many soils are stable when maintained under a gravel or crushed rock surfacing layer, but when free evaporation of moisture is cut off by an oil mat they become soft and spongy, due to capillary action and will not support traffic. This situation was illustrated in a startling manner and was reported to me as occurring on a section of highway in California on U. S. Highway 99 across the Imperial Valley located between Brawley and Coachella where the rainfall is generally approximately two and one half inches (2½″) per year. No vegetation of any kind will grow. A road mix was laid about 1930 or 1931 on this section of highway placing the mat on the desert floor. No investigation was made of the base since the area was so dry water was not considered a factor with which to contend. But in seven (7) years after the road mix was laid the road surface was a complete failure due to an unstable foundation. Free water was present under the bituminous mat. Investigation of the soil was made and found to be an A-7 soil, as designated by the Bureau of Public Roads, which we know is an extremely fine grained, dangerous clay. It required 7 years for this material to collect sufficient water, by capillary action, to become supersaturated, but once the water was drawn under the mat it never escaped. This is a characteristic of A-7 soils.

BITUMINOUS TYPES

The low cost bituminous surfaces adopted several years ago by the Highway Department as standard construction for contract work as well as small amounts of construction work performed by the maintenance department are, "Light Bituminous Surface Treatment," "Non-Skid Seal Treatments" and a 1½ inch Bituminous Surface Treatment "Road Mix" type.

(1) Light Bituminous Surface Treatment. This type is sometimes labeled "Blotter Type," "Inverted Penetration" or "Prime Coat Type" in other states but they all mean about the same thing.

On our primary highways the light bituminous surface treatment is employed to a large extent in a stage construction program by giving the roadway a prime coat which is used under heavier mats at a later date. The advantage gained by this method of procedure is that practically none of the surfacing material, placed on the roadway by the surfacing contractor is lost, while the roadbed is allowed to settle and assume its permanent position. An opportunity is also afforded to reinforce or repair weak areas before a more permanent type surface is constructed.

Where the surfacing material has been on the roadway for some time and is compacted, the preparation for the prime treatment consists of tight-blading the surface and, if necessary, wetting with water, then scarifying lightly in order to remove all irregularities. The roadway is then swept with a revolving power broom to remove all dust and fine particles that will prevent penetration of the bituminous cement.

In the case of new uncompacted surfacing, the roadway is brought to a smooth riding surface with motor patrol blades and rolled, then, as a final operation, a blade equipped with a wire mold board attachment is driven over the roadway just ahead of the distributor to give the surface a uniform texture and distribute segregated areas of fine aggregate so that streaking will not occur when the asphalt cement is applied. Sweeping of the new top surfacing material is not attempted.

By applying five-tenths (0.5) to sixty-five hundredths (0.65) gallon per square yard of MC-2 asphalt cement in two applications and covering with surfacing material as will be described later, a three-quarter inch (¾″) mat is formed on the roadway. These in many instances have lasted two or three years with only small amounts of patching and no additional treatment.

Two methods of covering the spreads of cement are defined in our current specifications. The first method provides for obtaining the cover material from the roadway by blading loose surfacing rock into two equal windrows on each edge of the roadway. One half of each windrow is spread back over the first spread of cement with blades and the remaining material in the windrows is used to cover the second spread of asphalt. Stockpiled top course material (¾″ to Dust) is usually provided to take care of the deficiency of aggregate in the windrow.

An optional method of covering the cement is provided by spreading all new top surfacing rock from trucks backed over the roadway. This method is preferred because of the even spread of aggregate on the surface, but is more expensive and occasionally the additional cost is not justified.

A five-day maintenance period is required between the two applications of bituminous cement as well as after the final spread for this construction during which time the contractor is required to broom or blade the finished surface, carrying excess material from shoulder to shoulder, covering all areas that are bare and will bleed and removing excess top course material from others. The process is to lightly broom the surface each day, leaving a light covering of fine aggregate evenly distributed over the roadway until the asphalt has cured and all danger of bleeding has passed.

The advantages gained by the use of MC-2 cement as compared with SC-2, formerly used, are believed to greatly outweigh the difference in cost of the two asphaltic cements, and there is a distinct advantage also by the use of the MC-2 material in that:

1. Once the MC-2 cement has cured and the maintenance period is completed, the roadway does not bleed during hot weather. This is especially true on the west coast where SC-2 treated roads have a tendency to bleed during the summer, destroying the non-skid surface.
2. A thicker mat is obtained due to the fact that MC-2 holds more surfacing rock in the roadway and in effect acts as a non-skid seal as well as a prime coat.

The density of traffic on our state highways makes it imperative that some form of bituminous mat be placed on the finished surfacing immediately after the grading and surfacing contractor completes his work. Clay fillers have been eliminated in the construction of the base and surfacing courses and it is therefore difficult to maintain untreated roadway surfaces during dry weather.

The light bituminous surface is, I believe, well suited for use by counties where a low cost surface is all that can be financed or is warranted by the traffic using the road.

Costs: The average cost per mile of 20-foot roadway, where 50% of the cover aggregate is bladed from the roadway and 50% is supplied from stockpiles, is approximately $1,000 per mile. Where 100% new coverstone is spread on the roadway the average cost is increased about $200 per mile. The above costs are based on contract unit bids and the aggregates were all crushed material. Where the counties have suitable sources and can produce their own aggregates with considerably cheaper labor costs, the price per mile can be somewhat reduced.

NON-SKID SEAL TREATMENTS

This type of construction is, as the name implies, used for re-sealing older types of bituminous surfaces that have become smooth or slippery for traffic, or porous and abraded. Seal coats are also used as wearing surfaces over the Light Bituminous Surface Treatment prime coat previously described where the foundation is good and it is desired to delay further stage construction of a heavier type of surface. When used for the last purpose, the Seal Coat is placed over the primed roadway usually not later than the following season.

Considerable development has been made the past two years in this Seal Coat treatment, in that the aggregates are now better graded and suited for the sealing operations and more care is taken to insure against ''kick off'' by traffic, of the coarse aggregate during the construction and immediately following the construction of the seal coat.

The non-skid seal consists of one application of from 0.25 to 0.30 gallon per square yard of bituminous cement on the prepared bituminous surface, followed by a spread of either three-quarters inch to one-quarter inch or one-half inch to one-quarter inch crushed rock or gravel, at the rate of from 120 to 140 cubic yards per mile of 20-foot roadway, which is equivalent to 25 to 35 pounds per square yard. RC-4 cutback asphalt is generally used. The rock is spread by trucks backing over the fresh cement with special spreader boxes attached or hung from the tail gate to insure an even, uniform spread of rock on the roadway. Next, the distributed rock is lightly broomed, by motor patrol blades equipped with wire broom moldboard attachment, in order to insure an even coverage of the surface with rock and the removal of any excess stone or ridges. The surface is then rolled and additional rock is spread where needed over bare areas. After final brooming and rolling, the roadway is dusted with a light spread of one-quarter inch to dust material spread at the rate of 35 cubic yards per mile of 20-foot roadway, or approximately eight to ten pounds per square yard. The purpose of the fine screening material is to keep traffic from picking up and throwing the coarse screenings that come in contact with the asphalt and cement and until the cement has cured sufficiently to securely hold the coarse screenings in place. During cool weather or when there is no ''pick-up'' due to light traffic conditions, the spread of ¼″ — 0 screenings is often delayed until the following day or entirely eliminated. The contractor is required to maintain the finished surface by brooming excess material from shoulder to shoulder of the roadway for a period of ten (10) days.

The non-skid seal produces an effective thickness of about one-half inch (½″). In some cases the Light Bituminous Surface Treatment with Non-skid Seal apparently will last, where favorably located as to climatic and foundation conditions, for many years and much longer than expected, under fairly heavy traffic.

Costs: The average cost the past season for the ¾″ — ¼″ seal coat was approximately $900 per mile of 20-foot roadway, while the ½″ — ¼″ seal coat cost approximately $745 per mile of 20-foot roadway. These costs are based on contract unit prices and in some cases the counties have and can cut them considerably.

BITUMINOUS SURFACE TREATMENT ''ROAD MIX'' TYPE

Where traffic conditions warrant, or the money is available and it is desired to construct a heavier mat on a roadway already settled and compacted, the road mix treatment is recommended for use. I believe a mat less than 1½″ thick should not be attempted. The highway department has laid some sections of this thickness on secondary highways with good success.

Satisfactory developments have been made within the last 2 years in the road mix type of construction. The former inability of obtaining uniformly graded aggregates by scarifying the existing roadway, and the resulting excessively rich or dry sections of mat, has been overcome by producing aggregates for the mat in two separate sizes and placing the coarse and fine aggregates on the roadway in correct proportions in order to produce a mixture that has suitable grading for the asphaltic cement to be used.

While the production of 100 per cent new aggregates for the road mix mat has increased the cost of this type, it is felt the added cost is warranted by the results obtained.

In many cases the roadway to be improved has been thoroughly compacted by from one to several years of traffic. To scarify this crushed rock or gravel mat would mean the loss of considerable stability. The addition of new aggregates consequently prevents the disturbing of the foundation and adds strength to the roadway. Oversize rock in the mat is reduced to a minimum and requires less forking during the mixing operations.

MC-2 asphalt cement has been used for the cementing medium in the majority of cases last season. This heavier grade of cement allows the use of coarser grading of aggregate and is more desirable than SC-2 cement formerly used. The MC-2 is less liable to bleed after the mat has been laid and produces a stronger and more durable mat. The use of MC-2 is advocated on all projects where the added cost of the cement warrants its being specified rather than SC-2.

Traveling machines for picking up the aggregates from windrows and mixing the aggregates and asphalt cement on the roadway are allowed under the specifications, but to date none of the contractors operating in this state have elected to construct the mat by this method. Motor patrol blades are used for the mixing and spreading operations.

Costs: "Road Mix" costs vary considerably with the location of the project and availability of aggregates. The average cost of one mile of road mix one and one half inches thick 20 feet wide is approximately $2650.00 which includes asphalt cement and production of all aggregates, as well as labor and equipment used in constructing the mat. Here again I believe the counties can better this figure in the cost of producing the aggregates.

Materials: I would like to comment briefly concerning the materials used in the types of treatments described. When investigating new and untried sources of aggregates a great deal of effort should be exercised along with a lot of good judgment in obtaining the best possible materials for each job with the least amount of haul. The new material and especially quarried rock should be investigated for its hydrophilic or hydrophobic properties or in other words whether or not the material will allow the asphalt film on the surface to strip off the rock in the presence of water. I used to think these were good seventy-five cent words concocted by someone to confuse the profession and had no real meaning to us in Washington, but I know now there are some sources of rock in this state that do have hydrophilic properties and are not good for bituminous types of construction.

In regard to some questions that have been asked and questions that some of you undoubtedly have in your minds regarding the use of screened, uncrushed aggregates for the bituminous types of surfaces discussed, it is very difficult to give a direct answer. It is my belief that each source of uncrushed gravel, whether it is for top surfacing material or is to be screened out for seal coat rock should be judged on the merits of each individual material. Generally, gravel that has a granular, sand paper texture and contains only a small percentage of flint-like, smooth, slippery pieces is acceptable and can be used successfully. I realize in many localities the use of such materials will effect enormous savings.

I wish to emphasize the need for competent and experienced supervision to successfully and satisfactorily construct the types of surfaces described. While the various operations involved in laying these types are quite simple, it is absolutely necessary that rigid and intelligent control be exercised at all times by experienced inspectors in all the steps of construction to get the results you will require and will be expected by your departments. We find that where the proper control is exercised whether you do the work by contract or with your own day labor forces, successful surfaces can be constructed.

In conclusion I would like to stress the following points in the construction of low cost bituminous surfaces:

1. Be sure the base or foundation is adequate and well drained. The bituminous mat will carry no more load than the foundation upon which it rests.
2. Use the best materials available consistent with the cost and use to which they are to be placed. Never use clay filler in any surfacing course to bind up the surface if you expect to oil the roadway.
3. Insist on competent and experienced inspection.

SECTION II

Fundamentals of Bituminous Construction

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PROF. W. J. EMMONS, Assoc. Prof. of Highway Engineering
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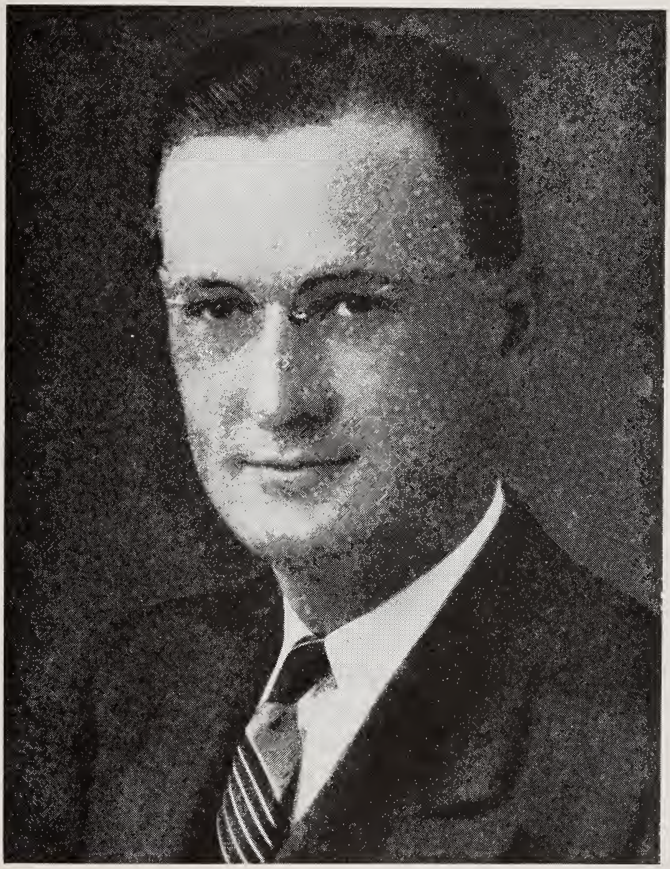
Ann Arbor, Michigan

Summary Report of Fundamentals Committee
Part I.

By

J. E. BUCHANAN
Dean, College of Engineering
University of Idaho



INTRODUCTION

The Fundamentals Section of this Conference is concerned with the principles of designing bituminous highways. Design principles are evolved or modified as a result of research. Subsequent construction methods may then be and usually are affected. In this respect the Fundamentals Section is closely related on the one hand to Research and on the other to Construction with only vague lines of demarcation between them. This relationship and the unique value of the Montana National Bituminous Conference was effectively stated by T. E. Stanton, Jr., Chairman of the 1938 Fundamentals Section as follows:

> "A great deal of valuable work has been done and is still being done by technologists and researchers in numerous laboratories throughout the country, but it must be realized that their efforts bear no fruit until they are actually applied on the job by some construction or maintenance engineer. And conversely, the laboratory man's efforts may be misspent and the findings of the researcher unimportant unless he is in close touch with the problems of the field engineer. It is in this bringing together of the research, design, construction and maintenance engineers that the Montana Bituminous Conference has been most outstanding."

In preparing individual reports and summaries the members of the Fundamentals Committee have intended to stay within their proper sphere and any invasions of either Research or Construction are unintentional.

For the purpose of this Conference the scope of "design of bituminous highways" has been assumed to include design of subgrade, subbase, base and surface. It is also concerned with principles of proportioning subbases, bases, and surfaces as to thickness and width for various subgrades, traffic conditions and climatic conditions.

The specific objectives set up for this Section of the 1939 Conference were:

1. A comprehensive review and presentation of design principles now accepted and being applied.
2. A report of all changes, improvements and trends within the past year.

Accordingly, individual reports were prepared by committee members and these constitute the source of information which has been consolidated for presentation in this summary. It is a matter of extreme pleasure to acknowledge and publicly thank all members of this Committee for the complete cooperation which was given. Praise is especially deserved, not alone for the quality of reports submitted, but especially because they were all prepared with limited notice during the peak of highway construction season. Unquestionably much of value in the individual papers has been lost in this consolidation. Because of necessity details are omitted, but fortunately subsequent discussion here and later publication of the papers will rectify this situation as well as any errors or misinterpretations by your reviewers.

The intelligent and sympathetic direction and coordination of D. A. McKinnon and D. L. Cheney of Montana paved the way and set the standard for committee work, and it is only hoped the performance of this Committee measures acceptably.

In planning the work of our Committee, cognizance was taken of the great nationwide interest in the subject of durability of asphaltic materials as it relates to fundamental principles of design, materials selection and combination. A special effort was made to review developments in this subject and bring it up to date for presentation here. This assignment was given to J. R. Benson of Kansas who did a monumental job of it. The summary of this Committee will, therefore, be presented in two parts: Part 1 by the Speaker, and Part 2 by J. R. Benson.

PRINCIPLES AND TRENDS IN DESIGN

General

Judging from the frequency of mention in reports and papers by individual members of the Fundamentals Committee the broad principle of designing bituminous highway "from the bottom up" is rapidly approaching the status of universal acceptance. This is indicated by the already widespread use and increasing adoption of soil surveys, selection of materials for subgrades, controlled compaction, better subbases and bases. It is also indicated by relatively little change noted in the design of bituminous surfaces, and especially by the increasing use of bituminous surface treatments. The latter reflects the belief that, except for heavy traffic, a base properly constructed on a well compacted subgrade requires a wearing course only thick enough to withstand traffic abrasion and to waterproof the base.

Subgrade

The use of soil surveys and the application of soil mechanics in designing the subgrade are either standard practice or are being tried experimentally quite generally throughout the country. Johnson of Iowa notes that when soil surveys show peat or muck near the surface or in considerable depth it is necessary to remove the material or relocate the road. The idea of relocation dictated by soil survey findings, while it has been reported before, is interesting. Extensive use of soil surveys for design data is also reported or indicated by Carter and Muir of Utah, Huston of Oklahoma, Stokstad of Michigan, and Lehmann of Louisiana. Cantrill reports that soil surveys are new in Kentucky, and Hallvik states that they became a part of preliminary work in all Idaho projects only this year. Gottlieb indicates that in Alabama tests are made during grading operations to assist in selection and to determine placement in the grade. Johnson of Iowa describes the use of a new field instrument, a penetrometer, used in soil surveys and subgrade studies of existing roads. "No claims are made for the instrument, but it is hoped that data may be collected by examination of failed areas, areas which show signs of approaching failure, and locations where no failures have occurred, so that a numerical value may be obtained for the subgrade resistance necessary to hold up various types of bases or surfaces."

The prevailing attitude toward selection and placement of soils in the subgrade seems to be that of utilizing scientifically and economically the material afforded by nature in the vicinity and avoiding soils and locations which experience has shown to be highly unsatisfactory. Selection is based upon various methods of test including Proctor analysis, Atterburg tests, California bearing and swell tests, linear shrinkage, and others. Apparently there is no standard specification or standard requirement for subgrade material. Cross-hauling is sometimes employed during grading to put best materials near the top. If necessary, select borrow is substituted in the upper one, two or three feet of the grade both in cuts and embankments. Stokstad relates that in frost heave areas in Michigan subgrades are tested in depth varying from 2½ to 4 feet.

Controlled compaction of materials going into embankment to 90 or 95 per cent of optimum density is reported by nearly all members who discussed this phase. Several also indicated that in cuts the soil below the final subgrade was loosened, brought to proper moisture content and compacted. Methods of field control of compaction are apparently not uniform, but some form of control is generally used. Proctor equipment is mentioned as well as sand displacement.

Other interesting items pertaining to subgrade design include: A sloped or crowned subgrade mentioned by Stokstad of Michigan and Hallvik of Idaho; excavation of adverse soils and replacement with select material at transition from cut to fill, and use of imported select material entirely for construction of grades in relatively flat, irrigated areas reported by Hallvik of Idaho. It is interesting to note that this latter procedure was previously described by Powers for certain situations in Arizona.

Subbase

Subbase is used on low quality subgrades to provide necessary support at the plane of the base course. Thickness of subbase is adjusted to offset subgrade deficiencies. The economy of Michigan designs using subbase was summarized by Stokstad as follows: ''Using costs based on current prices, it was found that approximately 3 inches of subbase could be placed for the cost of one inch of (chemically) stabilized gravel base course. By reducing the (chemically) stabilized base course thickness from 8 inches to 4 inches, it became possible therefore, to build a subbase of sand or pit-run gravel 12 inches thick without increasing the cost of construction. In other words, it was possible to construct a 16-inch granular blanket over the clay subgrade for the price of the old 8-inch gravel.'' Idaho practice as reported by Hallvik requires subbase material to meet rather definite specifications which stipulate grading limits, liquid limit, plastic index, and a Los Angeles wear limit.

Base

General application of the principle of designing roads ''from the bottom up'' has been mentioned: One ramification of this principle is an apparent tendency to standardize base courses as to quality and thickness which is justified by providing uniformly high supporting strength for the base through subgrade treatment and necessary thickness of subbase. From reports submitted there is considerable evidence that this is the case, and standardization on high quality bases is noted.

Alabama generally uses an 8-inch clay-silt-sand-gravel base built in two layers under very exacting specifications as to the quality of the fraction passing No. 10 sieve. Iowa uses both a ''rolled stone'' base constructed of crushed limestone and shale, and a soil-aggregate base. The latter is batch-mixed, spread and compacted under rigid control. Michigan, Oklahoma, Utah and Idaho also report similar high standards for base design.

An outstanding trend in base design is the reported attention paid to the fractions passing the No. 10 and No. 40 sieves. Maximum liquid limit and plastic index values of 25 to 30 and 3 to 8 respectively are common, and other limits such as linear shrinkage, volume change, and swells are frequently applied. Some states report a tendency to reduce allowable values of plastic index. Lehmann describes an interesting base constructed of "local iron ore gravel" in Louisiana which correlates with previously published research pointing to the desirable characteristics of iron ore in bases.

Stabilization

In the broad meaning of this overworked term stabilization it is being generally applied. In the restricted sense of stabilization by addition of chemicals, bituminous materials, Portland cement, etc., it is not very general, but is the subject of a number of experiments and experimental projects.

Stabilization of subgrades by proper drainage; by controlled compaction; by selection, combination and manipulation of natural materials is the general rule, as is stabilization of subbases and bases by mechanical means. The situation in Alabama was expressed by Gottlieb as follows: "We have made several investigations in regard to stabilization, and in our state it still appears that the use of local base course material, properly used and proportioned, is the most economical type of construction."

The use of materials other than clay, silt, sand and gravel or rock for stabilization of bases is not widespread, but is growing. Stokstad reports that Michigan uses a chemical addition in its plant mixed gravel bases. Cantrill states that Kentucky has about 60 miles of bituminous stabilized bases accomplished by treating soil-aggregate material as it exists on traffic-bound roads. New or experimental base stabilization projects are reported, described, or anticipated by Johnson of Iowa, Carter and Muir of Utah, Dent of Maryland, and Rowell of Vermont. Cantrill of Kentucky reports the use of an RC-sand mix. The once lively interest in sub-oiling seems to have cooled and the attitude now seems to be one of watchful waiting for time to prove results both technically and economically.

Bituminous Surfaces

Benson's questionnaire survey indicated that MC's are still generally favored for plantmix and roadmix, and RC's for surface treatments. Nevertheless a definite dual trend is noted toward the high viscosity SC's (SC-4, 5, 6) and toward plantmix using these SC's. Reasons for this dual trend include (1) some apprehension concerning durability of cutbacks, (2) greater use of dense-graded mixtures, (3) equipment development for mixing and placing. Emmons, Carter and Muir, and Hallvik mentioned developments utilizing high viscosity SC's, presumably SC-6, in Michigan, Utah and Idaho respectively. The trend toward softer AC's (higher penetration) is also well defined. At a previous Conference, Benedict observed that plantmixed, machine-laid, high viscosity SC mixes and higher penetration asphaltic concretes were approaching each other while retaining low-cost features, and Emmons of Michigan reiterates this view.

In opposition to the general trends just recorded, Blackwood of West Virginia reports a change to open-graded aggregate as does Rowell of Vermont. Committee members report more use of soft AC in surface treatments.

Not much new was reported concerning the design and control of bituminous mixtures. The use of stability tests (Hveem and Hubbard-Field) is spreading as is the use of adhesion-tension tests, such as the swell test and various stripping and washing tests. From reports of individual committee members, it appears that many agencies have fairly well standardized design and control measures and have confidence in them. This note of growing confidence specifically mentioned by Gottlieb and inferred by others seems significant for bituminous highways when a proper base and foundation have been built. The main attack at the present time is toward achieving greater durability and economy of wearing surfaces.

Mention has already been made of the increasing use of bituminous surface treatments (also called inverted penetration) for wearing courses. These differ somewhat in detail among states in an effort to achieve greatest economy consistent with performance using local materials, but they all follow the same general pattern. It is reported that Alabama has abandoned the drag seal described and discussed at previous Conferences and now uses a cold plantmix material instead. Kentucky has discontinued the use of petroleum naphtha liquefiers in cold-laid mixes and is using heavy MC instead. Cantrill also describes a number of experimental projects to develop economical surface treatments using local materials.

SEALING AND RESURFACING

Sealing is still regarded with varying attitudes. In general it appears that sealing is recognized as necessary to contribute durability to open mixtures and unnecessary on dense-graded surfaces. Bituminous surface treatments of pavements involving a cover of mineral aggregate to provide skid resistance and better light reflection are common and in some instances serve the true function of a seal as well.

Emmons reports that seals are not used on dense mixtures in Michigan, but that some counties and cities seal with RC and emulsified asphalt where farm traffic is a factor. In Utah sealing is now done with RC-4 whereas MC-4 was formerly used. In Idaho MC-4 is generally specified with RC-4 as an alternate when high quality cover coat rock is to be used; however, SC-3 and 4 are used when sand is the covering material. French reports that in North Dakota sealing is standard practice and generally is applied to finished bituminous mats after they have been smoothed and compacted by traffic. RC's are used and covered with 21 pounds of (½ - ⅛) inch mineral aggregate.

Resurfacing of old pavements of all kinds seems to follow methods in general use but with added attention to a leveling or smoothing course before applying the new surface. Gottlieb describes the resurfacing of an old Portland cement concrete pavement which was accomplished by first breaking the old concrete into pieces less than one foot square with a power hammer, then placing an 8-inch gravel base course, followed by an inverted penetration and a cold-laid plantmix seal.

THICKNESS

The determination of total thickness (combined subbase, base and surface) above subgrade is a problem which is much discussed but not yet satisfactorily answered. Kentucky assumes a 45° distribution of wheel loads, Utah uses Housel's theory, Iowa is experimenting with the "penetrometer" previously mentioned. Gottlieb of Alabama states that two important phases of design are still being accomplished by judgment:

1. Determination of thickness . . .
2. Deciding whether it is economically justifiable to place base course materials full width . . .

Hallvik presents an interesting summary of practice in Idaho based on three sets of empirical data collected by Public Roads Administration District No. 2, California and Arizona. District No. 2, PRA suggests total thickness above subgrade on the basis of liquid limit and plastic index roughly as follows:

Subgrade		Subbase, Base, Surface Total Thickness	
LL	PI		
Less than 25-35	Less than 6	0-6 in.	3 in. average
Less than 35	Less than 15	2-8 in.	6 in. average
Less than 50	Less than 30	9-18 in.	12 in. average
35 to more than 50	More than 30	12-24 in.	15 in. average

The California data suggest the following:

Subgrade Bearing Value Per Cent of Standard	Swell	Total Thickness Wearing Course Only
Above 50%	0-2%	
Above 30%	0-3%	5 in.
Above 20%	0-4%	6 in.
Above 10%	Above 6%	9 in.
Above 5%	Above 6%	15 in.
Above 3%	Above 6%	18 in.

Arizona procedure is in terms of linear shrinkage as follows:

Subgrade Linear Shrinkage	Total Thickness Wearing Course Only
0-2%	
2-4%	6 in.
4-6%	8 in.
6-9%	12 in.
Above 9%	15 to 24 in.

While these empirical tables are not entirely consistent, they point toward to solution which, when tempered with judgment and local experience, apparently gives satisfactory and economical design provided proper quality of subbase, base and wearing course are subsequently achieved.

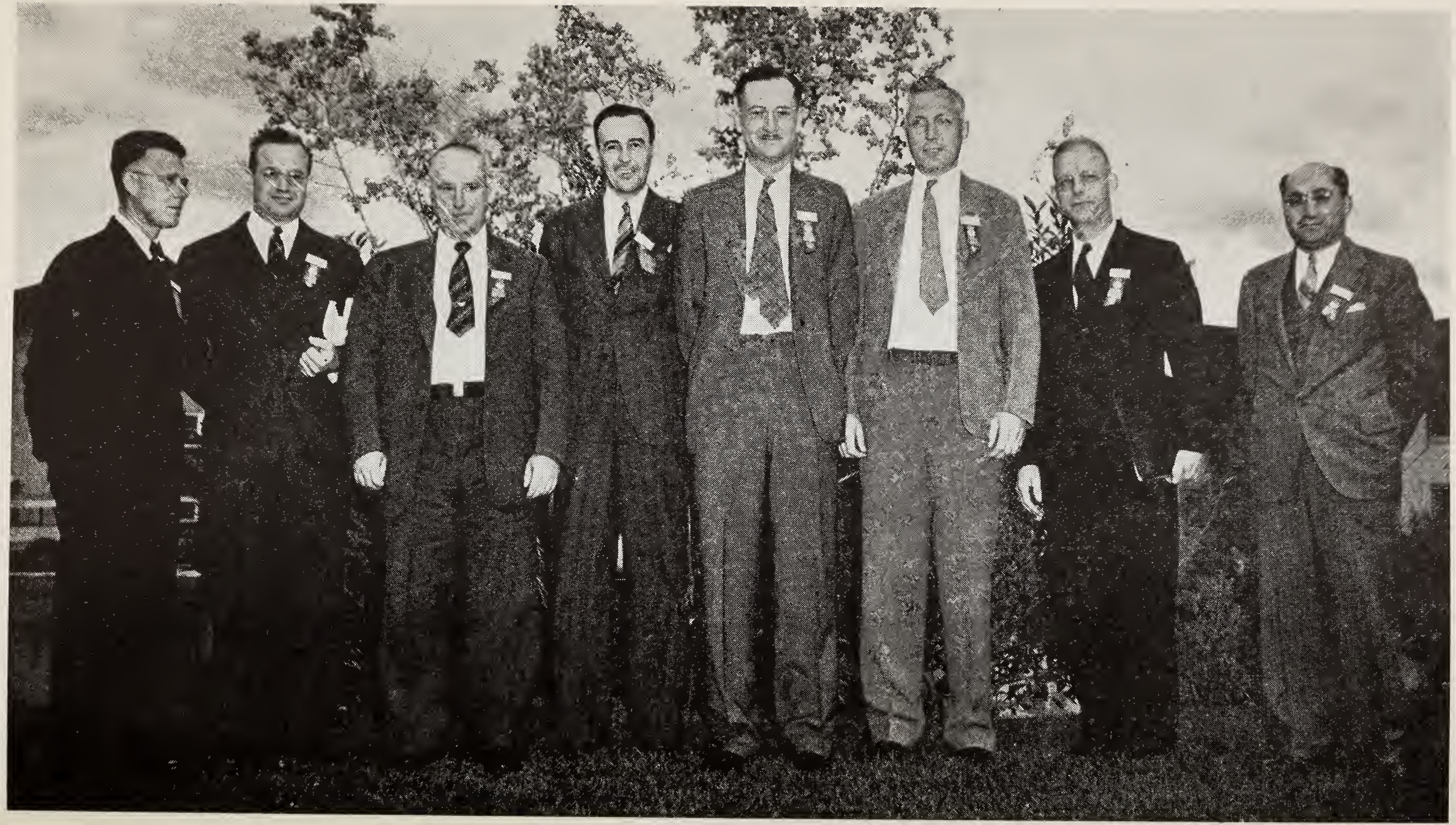
Full-width designs for subbase and base courses are commonly used, but there are some exceptions reported. However, if these courses are not constructed full width they are generally built wider than the wearing course to give edge support and frequently, as in Michigan designs, drainage facilities are provided for base and subbase less than full width. Western States seem to favor full-width designs except in cases where the shoulder material is sandy and permeable.

As a consequence of full-width construction of subbase and base, Idaho is discontinuing the thickened edge on roadmix and plantmix bituminous pavements, choosing instead to use the material that formerly went into the thickened edge to build a wider pavement.

SPECIFICATIONS FOR MATERIALS

Relatively few changes in materials specifications are reported. Aggregate gradations for bituminous construction have been revised somewhat to achieve either denser or more open mixes as desired. Abandonment of the Deval abrasion test and adoption of the Los Angeles test is a noteworthy change. Some form of adhesion-tension test has been added by a number of states, especially for aggregate to be used in surface treatments (inverted penetration) and seal cover. A washing or stripping test is used with various modifications. The swell test is still popular for dense-graded mixtures.

Specifications for asphaltic materials have been under close scrutiny and have undergone some changes. The principal objective seems to be greater durability. Since this is a very important subject, and one with many ramifications, a special survey and summary has been made by J. R. Benson for the Committee and will be presented as Part 2 of the Fundamentals Committee report.



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(Left to Right): James W. Johnson, J. R. Benson, Levi Muir, L. A. French, J. E. Buchanan, Prof. H. S. Carter, O. L. Stokstad, Leon Gottlieb.
(Mr. Hallvick not shown)

Conference on Fundamentals of Bituminous Construction

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L. A. FRENCH, Engineer of Tests
North Dakota Highway Department
Bismarck, North Dakota

Mr. Buchanan: Gentlemen, the period from now until adjournment is t be devoted to a discussion of fundamentals; that is, design of subgrade, subbase, base, bituminous surfaces, cros section, and discussion of design or durability as relate to design. In order to organize this discussion and confine it to some extent, the committee proposes to brea it up into subdivisions somewhat corresponding to th subdivisions presented in the summary.

The first subdivision pertains to design of subgrade To refresh your minds, some of the outstanding points in the summary were the use of soil surveys, the idea of relocation, description of a new field instrument known as the penetrometer, which is in the development stage in Iowa; selection of materials in design of subgrades based upon the Proctor test; Atterburg tests; plastic index; the California bearing and swell test and linear shrinkage; the substitution of select borrow on upper one, two and three feet of the subgrade both in cuts and embankments; the reworking and compaction of soil in cuts below the final subgrade; the use of Proctor equipment and sand displacement methods for field control; sloped and crowned subgrades; replacement of adverse soils with select material at transition from cut and fill.

Those are the points which were brought out in the summary pertaining to design of subgrade, and that phase is now open for discussion. Please do not be bashful. Does anyone have any questions he would like explained more in detail than was given in the summary? Maybe we are all holding fire for another subject that will come up later.

Some questions were submitted to me. By way of getting started, we might try one of them. The question relates to subgrade compaction practice in arid states where the cost of water is prohibitive. What do some of you do about that? Has anyone on the floor had experience on projects in arid regions, the design of which called for compaction, where the water was high? If so, how did you get around it other than just paying for the water? Any special features?

I think Utah has had some experience in that line. If there are no comments from the floor, I am going to call on Mr. Muir to make a statement or two about that phase of it. Will you do that, Mr. Muir?

Levi Muir
Utah

In the last year we have changed over from just grading any way without any compaction on many of our projects to a system where we require the material to be put down in eight-inch layers and compacted with the proper amount of moisture, tamped with rollers, and so on. There are certain points in Utah where it is absolutely impossible to get water. On those projects west of Salt Lake, on U. S. No. 40, water had to be hauled in by train to supply the contractor's camp, and so on. It is obvious that if we go to work in a country like that we just have to forget the use of water. In some cases we might be able to secure really good compaction by the use of more rolling, but I think on projects of that kind we just have to abandon the use of water.

We had one project of a similar nature that was opened for bids late last fall and contracted. The contractor moved in, in January, and we found by being careful of the time we rolled and by making use of the moisture that fell from the storms we were able to get equally as good results; in fact I think we got better results than had we built that road in the summer time and hauled the water. The water haul would probably have had to be about twenty miles. I am sure in Utah where the nearest water is great distances from the project we will just have to abandon this procedure, unless we make use of the moisture that falls. So far, on the one project it worked out very well.

W. S. Housel: Michigan

Mr. Chairman, I would like to ask Mr. Muir what densities he is able to get when compacting practically dry as compared to the standard density at optimum moisture content.

Levi Muir Utah

On projects where the moisture content happens to be within 2 or 3 per cent of optimum, extra rolling with the tamping rollers will produce satisfactory compaction. This is especially true if the Proctor curve happens to be rather flat as we sometimes find with Utah soils. Frequently the moisture content of the soils is 5 to 10 per cent below that required for optimum compaction. In these cases it is impossible to anywhere near approach optimum compaction. We checked a case where the moisture content and rolling had not been given close attention and found the compaction to be only 57 per cent of optimum. I think this explodes the theory that contractors and field engineers often advance that the grading machinery will give equally as good compaction as the tamping rollers. Any system that does not take into account the proper amount of moisture in the soil is bound to give indifferent results.

W. S. Housel: Michigan

Those are fine-grained soils you are talking about?

Levi-Muir: Utah

Those are silty soils, soils that have dry weights of about 105 to possibly 110 pounds per cubic foot. I think the thought that contractors advance, that the grading machinery will roll earth equally as well as the tamping rollers, might be partly right, providing the material has the right amount of moisture, but if there is no attention given to the amount of moisture, I don't think any system will anywhere approach an optimum compaction.

V. G. Gould:
Iowa

I am interested in knowing from Mr. Muir or someone from any other state whether in your attempt to use controlled densities, if you attempt to control that density for the full depth of embankment. I am thinking particularly of embankments of ten feet, and in excess of that. Do you attempt to control the densities for the full-depth of the embankment?

Mr. Buchanan:

Will anyone volunteer? Mr. Hallvik, what do they do in Idaho?

Cliff Hallvik:
Idaho

It is new in Idaho, but I think we give greater importance to the density on the top three or four feet of the fill rather than down below. We have tried to control it as much as possible below, but it is new and we haven't been able to get good field controls yet. There is too much dirt-moving equipment on most jobs.

Leon Gottlieb:
Alabama

We control the compaction of fills from the bottom layer all the way up the fill to top layer. We require a minimum compaction on each layer of not less than 90% of the theoretical density as determined by laboratory test in accordance with A.A.S.H.O. test method.

O. L. Stokstad:
Michigan

We compact from the bottom up as they do in Alabama. Professor Housel might check me on that.

W. S. Housel:
Michigan

Yes, that is correct. We compact the clay fill and have control of the moisture content and density for the full depth of the fill.

That brings up another point that might be somewhat interesting. In the clay fills, where they are apt to be subjected to flooding, in drainage courses or on low land, we have made a practice of putting in a sand layer on the bottom as a cut-off for capillary water, a sort of drainage layer, and then start the Proctor fill on top of the sand layer. Whenever the clay is going in, we control the density for the entire depth of the fill.

My reaction would be that it would be rather important that you did do that, because stability in the lower part of the fill would suffer if you didn't. Over a period of time, if you left the lower part uncompacted, you would get settlement and subsidence of the upper part. Thinking of fill construction in general, I would think it might destroy the value of the upper part if the lower part isn't compacted.

L. A. French:
North Dakota

We compact in layers from the bottom up.

Levi Muir:
Utah

We are compacting from the bottom up. We have one project proposed now where a lake created by a reservoir is to come in contact with the highway line at three or four points. We are specifying there the use of rock in part of the fill to five feet above the water line. We have, I think, a slope of 3:1. We are following somewhat the system the Reclamation Service uses in building dams. Then we start from that point up, and of course compact the material.

There is a thought that has come to my mind since this discussion started on the amount of water used in very high fills, like California is using. We have compacted one 80 feet high this year. Mr. Porter of California intimated to me that in his opinion the amount of water required by the Proctor test was probably quite excessive in some of their very high fills, and there would be great danger of the fill not having stability with the excessive weight, which would probably cause some trouble. I would like to hear that discussed.

Mr. Buchanan:

Is there any discussion on the use of the Proctor test and the total amount of water? Mr. Gould, does that give you a sufficient cross section?

V. G. Gould:
Iowa

I would like to hear from Mr. Benson, and the other gentleman, if they have anything to offer.

Mr. Buchanan:

Mr. Carter is also from Utah, so we will assume Mr. Muir spoke for that state.

J. R. Benson:
Kansas

Mr. Keeling, Engineer of Construction of the Kansas Highway Commission, is present and I would like to refer that question to him for reply.

R. C. Keeling:
Kansas

We compact from the bottom up, the same density all the way.

K. C. Vail:
Colorado

I would like to ask Mr. Housel how thick the sand layer is he put on the bottom to drain the clay.

W. S. Housel:
Michigan

We put the sand layer in so it will be about two feet above the highest possible point of flooding. That is just a guess, of course, but we figure it should be enough above the probable water level so that there would be little opportunity for capillary moisture to get up through the sand into the clay.

While I am on my feet I might add something to Mr. Muir's comment. Mr. Porter, of the California State Highway Department, was in Ann Arbor a couple of weeks ago, and we had some time to discuss methods of fill compaction. They are not using the Proctor density, but are compacting at lower moisture contents and are compacting under heavier pressures and more rolling to produce a considerably higher density than the Proctor. They go up to 110 and 120 per cent Proctor. Compacting up that far, if we had the optimum moistur as defined by Proctor, the voids would be overfilled an would result in considerable instability. On the high fill in California, according to Mr. Porter, the stability requirements are much higher, and the change of stability fo a very small change of density in that range is very important to them. They keep the water out as much a they can, and do everything they can to prevent wate from getting to the compacted fill afterward.

V. G. Gould: Iowa

I do not care to use more time than this subject deserves, but there is one matter in connection with fill compaction that it seems to me is of vital importance in connection with compaction from the bottom up, and that is the moisture control necessary for any specified density you are trying to secure. Moisture control is simple enough and can be handled satisfactorily when soils are dry to begin with, or relatively dry, but how do you handle the matter of moisture control without reducing the rate of production of the contracting forces with extremely high moisture contents, which I know some of these other states have just the same as we have in Iowa.

In the fall, we will say, when drying from the air is very, very slow, moisture control may reduce the yardage covered in any day's or month's time to that which will fall into the old grading standards of quantity rather than the up-to-date standards of grading, which we know result in tremendous quantities of earth per day. How is excess moisture handled on such work?

W. S. Housel: Michigan

I will try to answer Mr. Gould's question. We had a rather sad experience with a project where the moisture content was high. Unfortunately, the project got under way in the fall of the year and we simply couldn't get the moisture out of it. It was a headache all the way through, and the project had to be put over. It couldn't be done in the fall.

Our experience on that project has led the department to establish several "don'ts" in fill compaction, using clay fills, and one of them has to do with a period of year when a Proctor fill should be set up. I believe the practice established was not to set up any clay fill compaction except between May 1st and September 30th. Any construction outside of those dates we wouldn't set up as Proctor fill; another safeguard was to investigate the moisture content of the borrow material and make sure it wasn't so high it couldn't be dried out. In any event, we should make sure the contractor knows what he is getting hold of before he gets the job, because they simply have a terrible time and can't get effective compaction under unfavorable weather conditions.

That brings up another point that appears to be of some importance. We are rather fearful of over-compacting some of these clay fills. That is, if the clay in its natural state has a moisture content that is higher than the optimum for the Proctor fill, and you reduce it to the optimum and then compact it beyond its natural state, over a period of years it appears there is a possibility that the natural moisture content will tend to go back in the soil and produce swelling and stresses in the fill which may be much more dangerous than if we hadn't tried to get that maximum density. Possibly the emphasis should be put on getting a uniform density and a density that will give sufficient stability, but not necessarily the maximum density.

For that reason, we don't set any definite density in the specifications, but would rather let that be decided on the job, so that if we don't want to compact to the maximum Proctor density we can use any density we think is the right one. Mr. Schaub, is that correct?

J. G. Schaub: Michigan

Yes.

W. S. Housel: Michigan

I know there is some objection to not specifying density directly, but that seemed to us to be the wisest way to handle it.

L. C. Campbell: New Mexico

With reference to compaction in the absence of water, of which we have considerable scarcity particularly in the southern part, the only thing we can do when water is not feasibly available is to use such methods as have been used extensively before. For quite a number of years New Mexico has tried to secure its compaction through those methods, principally by directing the contractor's equipment over the fill while it is being built, building it in thin layers and rolling each layer, sometimes supplementing the contractor's equipment with rolling of one kind or another, and compacting from the bottom up.

Mr. Buchanan:

The time is getting along. Unless you insist on some further discussion of this subject, we will move to the next subdivision, subbase and bases.

To refresh your mind on the summary previously presented, some of the points were as follows: The idea of standardizing upon uniformly high quality bases, standardization as to thickness and taking care of variations by subgrade treatment and subbase variation. The question of maximum values of liquid limit and plastic index may properly come at this time.

The values reported from the papers reviewed were: Liquid limit 25 to 30 maximum; plastic index, 3 to 8 maximum. Other tests may also be used, such as linear shrinkage, volume change and swell. Stabilization may properly be included in this, either the broad definition of stabilization that Dr. Winterkorn gave us yesterday, or the restricted definition of stabilization involving the use of chemicals, bituminous materials, cement, or other admixtures. Subbases, bases, and stabilization are then open for discussion.

C. E. Shumate: Colorado — I would like to know if anyone in this region has followed the formula advocated by Messrs. Willis and Hogentogler of the Public Roads Administration as to the depth of base required based on the soil analysis of the subgrade. As I recall, they had a very definite formula showing the base requirements from about four inches up to twenty-four inches, based on the analysis of the subgrade soil. I believe both Arizona and Ohio have formulas that are similar to that, and I just wondered if anyone in this particular region had used any of those formulas.

Mr. Buchanan: Can anyone supply information in response to Mr. Shumate's question?

I. E. Russell: Wyoming — We are using what is known, I believe, as District 2 formula, based on liquid limit and P.I. We are using that in our design at the present time.

Mr. Buchanan: Public Roads Administration District 2 equation, based upon liquid limit and plastic index, is used in Wyoming. I believe it is also used in Idaho; at least it was reported by Mr. Hallvik in his paper.

Are there comments from any other states or any other questions or discussion on the subject of subbase, base and stabilization?

Is there any disagreement as to maximum permissible values of liquid limit and plastic index? The range in maximum permissible values as reported was 3 to 8. I believe some agencies stupulate 6. Are there any opinions or violent disagreements with that range?

Leon Gottlieb: Alabama — I would like to add a comment on the liquid limit and plastic index. In connection with the Bureau of Public Roads, we sampled base courses four years old and older in an effort to determine the maximum limits for the L.L. and P.I. based on the observation of the bases that were giving satisfactory service. We found in general that an L.L. of 25 and a P.I. of 6 were the proper limits, except that we could permit an L.L. of 35 and a P.I. as high as 15, provided the material showed low linear shrinkage and volume change. We recently changed our specifications to require a maximum limit of 25 for L.L. and 6 for the P.I., but permit base course materials to be used if they run over those figures provided the linear shrinkage and the volume change figured from a moisture content equal to the field moisture equivalent are within reason, for this we require a maximum linear shrinkage of 3.2 and volume change of 10. If they are within those limits, we permit material to be used which have P.I.'s above 6.

Mr. Buchanan: Is there any other discussion? There were some questions submitted to me. I can run through one or two of these. If they strike a responsive chord, all right; if not, we can move on to another subject.

One of these relates to the relative importance of compacting and curing with respect to final base stability. Will anyone offer any comments on that question?

Here is another one: What is the safe percentage of moisture in bituminous or soil aggregate bases before applying the bituminous surfacing? Do any of you have any definite limits in your specifications, or are you applying any such limits in your work?

L. C. Campbell: New Mexico

We have a limit of two per cent. Of course, the critical moisture would depend upon the type of material which it is in, but we have a two per cent limit.

Mr. Buchanan: Thank you. Are there any other comments?

Another question which was proposed, on which some information is desired is this: What is the effect, or are MC primes satisfactory on stabilized bases? Does anyone care to comment on that question?

J. G. Schaub: Michigan

We use primes under our oil aggregate construction on stabilized gravel bases, and as a general rule use MC-1. We have a slightly different specification than The Asphalt Institute, designated as MC-1B. We do use MC prime on stabilized bases. We also use tar, TC-2.

L. C. Campbell: New Mexico

We also use an MC-1 prime. In some cases it is quite stabilizing; in other cases, while it does add to the stability, it may not be so highly successful. That does not mean it is a failure at all, but it is just the difference in the range of results using that material on the various aggregates that are used.

I. E. Russell: Wyoming

On practically all our projects we specify MC-1 prime, but recently we have attempted to cut down as much as possible the clay or soil binder in our courses. If we have a more or less open base because of that, then we go to an MC-2 or even an SC-2 for our prime material.

Mr. Buchanan: Do any of the committee have any remarks on that?

The next subdivision refers to design of bituminous surfaces, and that phase or division of this committee's work is now open for discussion.

V. A. Endersby: California. A point that might be worth bringing up is a relationship between the bituminous surface and the subgrade which may sometimes be overlooked. The stability of bituminous material, especially in thin layers, depends not only on its own internal characteristics but on the binding power of the material on top and on the bottom of it. That is to say, if you put a rubber sheet under a short sample which is being given a compression test, you will reduce its bearing power greatly. If in practice you have too heavy a prime coat, producing a slippage plane between the bituminous surface and the subgrade, you will unstabilize the bituminous pavement regardless of any subsidence of the subgrade. That is probably also true if you have lack of local adhesion in the base. That is, you might have a base which will permit a slippage of the bituminous material along the base, by separation of particles of the subgrade, for instance, although the base itself is strong enough to carry the load. Such phenomena as that may account in some local cases for some failures by shoving where the base is nevertheless strong enough to carry the load.

V. G. Gould: Iowa. I would like to ask one question regarding moisture content of emulsion mats at the time of compaction. In explaining that question just a little, we know that in order to have proper moisture content for dispersion of emulsion in the material, a moisture content somewhere around 8 per cent is necessary, which is relatively high moisture content, apparently, for compaction. If that is true, what moisture content is considered satisfactory for the initial compaction of the emulsion mat?

W. H. McCallie: Montana. In specifications by the American Bitumuls Company as written by Mr. C. L. McKesson, there is no moisture content given, as such, but it does say that a roller shall not be put on until such a time as it will not shove the mat under the roller or distort other than might reasonably be expected in rolling if it were dry. In the case of emulsified asphalt stabilized soil bases, the moisture content should be reduced to a 5 per cent maximum before surfacing with a mat.

W. S. Housel: Michigan. The question Mr. Gould has asked strikes me as a very important one, and one that certainly in present soil stabilization practice involves considerable difference of opinion.

In the case of asphaltic emulsions there is emphasis on the necessity of a drying period to obtain proper stability. In the case of some of the other materials that use high moisture content for dispersion, such as asphaltic oils and tars, there is a feeling on the part of the producers that the mixture may be compacted with a relatively high moisture content. They also feel that the road may be sealed at a relatively high moisture content.

From my contact with that work, I have come to the general opinion that the mixture should be gotten just as dry as possible before it is sealed, even though it may be compacted at a moisture content that is relatively high.

It seems to me the drying period is such an important factor in the proper distribution of the bitumen in the soil structure itself that it is a dangerous procedure to seal that mat until it has had every opportunity to dry out. I think the question, though, is in a very controversial stage now, and perhaps the development of the next year or two will give us some more definite information. Certainly, from the standpoint of laboratory tests, the drying period is a most vital step in the procedure of soil stabilization, and until we have information to the contrary I feel inclined to be guided by that information.

On the airport surfaces the ones that have given trouble in soil stabilization are those surfaces in which the total liquid content was up to the danger point that may overfill the voids. I really think that in the next few years, when we have more experience in soil stabilization and serviceability of that type of surface, more and more emphasis will be put on drying out the mat, getting just as much moisture out of it as possible before it is sealed.

H. H. Houk:
Washington, D. C.

I would like to urge everyone contemplating soil stabilization in any project, be it airport or highway, to first investigate the possibilities, the feasibility and the economy of physical soil stabilization, using natural materials available in the vicinity without the addition of asphalt, tars, or chemicals that may cause soil reactions of which we have no knowledge, and which at the present time is very difficult to obtain for the different soils.

H. G. Nevitt:
Missouri

I might make a comment in connection with aggregate asphalt mixtures, on a point discussed yesterday by Mr. Endersby. I am not sure I understood him, and I thought I would just make a comment on it. He was discussing Hveem's method of testing stability of 140° F. as a design control for optimum asphalt content, and commented that asphalt at this temperature had no appreciable viscosity, and therefore you didn't get the effect of the relative asphalt penetration, and so on, at lower temperatures.

I don't believe that is Hveem's reason for testing at that temperature, nor ours in the work we are doing. We do not test at 140° for full information, because we are a little inclined to go along with Vokac in the more complete meaning of the 77° test for use purposes. But if you test with 140°, you are testing at the point where you have your maximum expansion of asphalt. In other words, your asphalt expands more rapidly than your aggregate, and will show the greatest actual volume content of the mix at this temperature.

To avoid the very point Professor Housel made on soils, namely, not to overfill your voids, you need to test at 140°, in our opinion, whether it is a stability test, a compression test, or some other, merely to make certain your 140° temperature permits overfilling of the voids and whether or not that gives you all the information you need. You should, in addition, test at, say, 77°, for use purposes is something we haven't decided upon yet. But I do think the 140° temperature of test is quite critical in this particular way, especially on aggregate mixes where there is some danger, if you get the proper asphalt content, of getting near the void overfilling point at higher temperatures.

W. S. Housel: Michigan

There is one point I didn't make, in answer to Mr. Gould's question, which is I think very important, and that is the primary function of soil stabilization with bituminous material is the waterproofing of the soil structure to prevent the ready access of moisture and loss of stability later. Unless the bitumen is properly distributed to seal the soil structure, it can vary much more readily and allow moisture absorption to take place. That is along the line Dr. Winterkorn spoke yesterday about controlled moisture content. If you once get the bitumen particles coated, or the bitumen distributed to seal the capillaries, that mixture is very much improved over what it will be if the moisture is left in there and the bitumen hasn't been carried into its most effective locations in the soil structure.

Along that point, the paper which Mr. Brown presented at last year's Conference—I believe it is in the Proceedings—gives results of water absorption tests on stabilized soil mixtures or on using an asphaltic material, and shows the tremendous difference in the absorptive properties of the material that has been dried over that which has not been dried. That, I think, is probably even more important than the loss of stability from the moisture, and emphasizes again the importance of drying that moisture out some time before it is finally sealed up.

Ir. Buchanan: Are there any other comments? We are using our time and are just about on schedule.

If there is no further discussion on the question of design of bitumen surfaces, the next subject is sealing and resurfacing. There was a time in these Conferences when sealing was a very actively discussed subject. Are there any comments or questions concerning sealing and resurfacing?

We can let that one rest for a minute, if you like, and next discuss the subject of design of cross section as to thickness and width. To refresh your minds and review the summary given, some states reported the use of the assumption of 45 degree distribution of wheel loads; some reported the use of Housel's theory; Iowa reported the development of a new instrument, the penetrometer; several authors reported thickness and width were entirely matters of opinion or judgment. The data of Public Roads Administration District 2, the California bearing and swell tests, and the Arizona linear shrinkage data were presented.

Are there any comments or questions, or discussion on the subject of design of cross section? There was a question asked, if someone in the audience will supply discussion. This question pertains to the use of asphalt membrane treatment over the subgrade soils instead of sand blankets. Has anyone had any experience with the use of asphalt membrane treatments over subgrade soils prior to the construction of subbase and base courses?

[. E. Russell:
Wyoming

We had just one project in which we tried that treatment, but it didn't turn out too successfully. I believe we got more advantage from the treatment down the slopes than we did over the grade. We did cut off some erosion, but we didn't stop our frost. One of the reasons we used the treatment was to try to stop capillary water.

Mr. Buchanan: Could you describe the treatment for the benefit of the group?

I. E. Russell:
Wyoming

We used a double-shot treatment of RC-2 and sand, just spread on the road and covered with sand, and followed with another of the same type.

Mr. Buchanan: Do you recall the quantities of RC?

I. E. Russell:
Wyoming

I don't believe I can. I imagine probably in the neighborhood of one half gallon to the square yard on each application would be pretty close.

C. F. Seifried:
Wyoming

I might give a little intimate history of this proposition Mr. Russell was talking about. The California Highway Department, as you know, experimented on this proposition and it was written up in some of the magazines. We in Wyoming thought it might cure the trouble we are having in certain isolated places where we have highly colloidal soils. We call them our Bentonite bumps in a practical way. Fortunately, they don't extend over any very great length, but they do come up in the spring. I don't think they are frost boils; they may be, but this Bentonite, as you know, absorbs just enough moisture that in the spring it makes bumps on the road. After a month or so, in the summer, they go down.

We read the report on what California was doing and thought we would try the treatment and perhaps stop the bumps by keeping out the moisture. We have practically no precipitation at all, less than ten inches, but from some place or other enough moisture gets in to swell the stuff up.

Last March I was driving over a piece of road where the experiment was tried in southern California. Some of you may be familiar with it. The engineer who had me along told me how they had insulated and put the film in, and he said we were driving over it then. He said, "It is very bad adobe soil, and this mile on which we used the film,"—I think it was a mile, anyway a short stretch —"has certainly been a great success. We haven't had any trouble with it." He stopped a minute, and then said, "We are now driving over that same soil, and you can see the condition is just the same. We have had practically the same success where we didn't use this film at all."

Mr. Buchanan: Are there any other comments? If not, we can come back to any of these, if there is any time left, provided we get over all the subjects; the next subdivision pertains to specifications for materials. One question which has been presented refers to the use of stripping tests—I presume by that is meant tests which involve some agitation—for aggregate to be used in dense graded mats on a primed and stabilized base, which mats are then subsequently sealed. Does anyone have an opinion as to the usefulness of a stripping test for aggregates to be so used?

It is my understanding that the swell test is quite generally used for such purposes, although there has been some advocation of the stripping test. The question is whether or not a stripping test involving agitation is applicable to a dense graded mat.

H. G. Nevitt:
Missouri

Perhaps my remarks are a little out of line. I just wanted to say this matter of stripping in general, rather than specification, is something that seems to be getting a little different slant. From visiting around in the past years I think there is a little change in attitude. They used to think stripping was something that happened on a few soils or aggregates, where the asphalt completely disappeared from the surface of the aggregate and you had a nice white, water-bound macadam left, or something very near that. Now I believe there is a tendency to think that stripping, or water infiltration of that type, is a more frequent cause of minor maintenance than was thought. You may have a few spots of the aggregate stripping, due to stripping mineral constituents, which cause a certain amount of water to get in the road and result in corrugation in some places, or other troubles.

I just wanted to bring that up. I don't know that it has any specification meaning. I think we are getting to the point of thinking that stripping in a mild way is more serious than we previously thought in that it does affect maintenance.

Mr. Buchanan: Are there any other comments pertaining to any subject under the heading of specifications for materials? If not, we have reserved a fair amount of time for discussion of the subject of durability, under which heading will come the use of positive and negative materials, the use of the Oliensis test or any other test, design of mixtures to achieve durability, or anything under that general heading. That subject is now open for discussion.

H. G. Nevitt: Missouri

I want to ask a question of Mr. Benson, because he didn't spend much time on it.

As I understood his paper, where he was comparing positive and negative materials he had a slight difference in the results on the surface treatments. He showed a ratio of good and average to bad, but on the right hand side of the chart he showed a third diagram. He had negative and positive, and had a third which was positive film, which I understood meant positive clear film. That showed all good or average, and no bad, which I understood him to say were the results of his data. I would like a little amplification of that, because I think the film test, as an addition to the positive test, has considerable interest.

J. R. Benson: Kansas

As most of you know, about two years ago we presented several papers on work which we had performed in developing a film test on asphaltic materials. While there are several reactions which may be observed in testing films of asphaltic material, only the reactions of "clear and coagulated" have been considered in experimental construction. For such construction, an asphalt specification was set up which required, of positive Oliensis material, a clear film at the end of 48 hours' exposure to air at a temperature of 200° F.

Several penetration-armor coat surface treatment projects were constructed in 1938 employing this film specification. Other projects frequently used one or more cars of asphalt meeting film specifications, since the latter also met the requirements for the standard positive Oliensis asphaltic material.

In making the survey of surface treatments, several areas employing film test material were included. The information on the behavior of the film test can be considered only as an indication; definite conclusions cannot yet be made. It is indicated that there is a difference in the behavior of the positive Oliensis material meeting the film test and the positive material which will not meet the film test. It is entirely too early to make any really definite statement on what the test will do. The results noted are indications only, with the test applying, as I stated, only to the positive Oliensis materials. In our experience practically all of the negative Oliensis materials will meet any film test as to "clear" film which may be applied.

I might amplify just a little bit here. A while ago I presented a chart in which I tried to show you a number of different methods or ways in which asphalt might change and decrease in serviceability. I think a great deal of confusion in the use of these various predicting tests could be eliminated or reduced if we understood the various methods by which asphalt changes. As a rule, tests which we apply to asphalt tend to control largely only one reaction, but unfortunately we have a great many more reactions than just one. That, in turn, comes back to the general proposition of one group, one area, one organization, one field, or one producer. Whatever the particular case may be, whatever difficulty you are having, there are a number of things that enter in. While one test may prove a very beautiful solution to whatever problem you may have in your particular area, type of material, or field from which you are obtaining material, and so forth, that may not apply to other places.

Also, in connection with this film test, the test in our estimation—it is still open to argument—controls largely just one, you might say, of the reactions which are occurring in the asphalt, and we believe that could be defined as colloidal instability or a breakdown of the phases of the colloidal structure of the asphalt. There is nothing there that will tell you the material is immune to rapid change of volatilization or by thixotropy, external chemicals, or any of the other reactions. It merely controls one factor. Where that one factor is the predominant cause of difficulty the application of this test might form a solution, but in view of the many other things which are coming in which must be considered, a very careful analysis must be made of the behavior of the materials.

I believe much good could be done if all of us could perhaps understand a little more about these various reactions which are occurring in asphalt, and the relationship of these tests which we propose and the actual behavior of the material. There is a great deal which we could observe and learn if we could combine our knowledge of predicting tests and the reactions which are occurring in our asphalt. I think ultimately we will arrive there, but at present there is quite a bit of confusion.

Does that answer your question, Mr. Nevitt?

I. G. Nevitt:
Aissouri

Yes.

Mr. Buchanan:

Are there other comments? I know there are some people here who are interested in a brief explanation of the Oliensis test. Mr. Benson, would you give us a very brief description of the Oliensis test as it is applied?

. R. Benson:
Kansas

I didn't think there was any possibility of anyone not knowing all about that.

The test consists essentially of dissolving or dispersing, with or without the aid of mild heat, 2 cc. of the asphaltic material into 10.2 cc. of a particular distillate solvent. The procedure, apparatus, and solvent are carefully controlled. After a period of setting, a drop of the stirred solution or dispersion is placed on a designated type of filter paper. If the resulting spot has a uniform color, the material is designated as negative. If, however, a darker central area or an annulus appears within the spot proper, the material is designated as positive.

The test is really a form of solubility test. It is somewhat similar, for instance, to an 86° A.P.I. Naphtha solubility and a CS^2 solubility. In the former case, all of the asphaltenes (by definition) are precipitated while with CS^2, no asphaltenes are precipitated and all of the bitumen is soluble. The Oliensis test falls between these two extremes and by careful control of test conditions separates asphaltic materials into two classes, one in which no precipitation occurs, designated as negative, and the other in which a precipitation does occur, and is designated as positive. The test to a large degree is presumed to indicate the colloidal stability of the asphalt, although as to the degree in which that is determined, I would not like to say.

H. G. Nevitt:
Missouri

I would like to get a couple of points on the record in regard to the Oliensis test. In California, Dr. Bray, formerly of the Union Oil Company and probably one of the outstanding men on solvents for heavy petroleum products, emphasized what Mr. Benson has just said, that the Oliensis test is not a test to show whether the material is homogeneous. It has no relation to that. It simply shows its solubility in a certain solvent.

That comment was followed up by Mr. van Doormaal of the Shell people, who further added that the so-called positive materials or materials which Oliensis says are heterogeneous rather than homogeneous are actually, by physical tests, shown to be homogeneous, if there is any difference. The so-called Oliensis heterogeneous materials are actually the most homogeneous products. If you are going to say a material is not satisfactory because it is positive to the Oliensis test, you have to make some accounting for the very wide spread satisfactory use of tars, which are very highly positive, and which are nothing more than a positive type of asphalt. They are hydrocarbons, just like apshalts are.

So any explanations of quality for rejecting materials which are positive to the Oliensis test, or taking the attitude that they are not satisfactory, must contain also an explanation of why tars, which are essentially material of that type, have such a satisfactory record, although not universally satisfactory. Of course there are good tars and poor tars.

Mr. Buchanan: Gentlemen, we have about ten more minutes. Discussion on this subject or any other of the subdivisions of the Fundamentals Committee is in order.

R. M. Schwegler:
Oregon

I would like to reopen the subject of stripping. It seems to me there is rather poor correlation between our stripping test and actual field behavior. In other words, I am going to question the real value of our present stripping test. Our experience, in District 1 of the Public Roads Administration, is that about 50 per cent of our aggregate shows either fair or poor stripping. This test has been run for several years. We have never found any percentage of mat failures attributed to stripping which has run in any sense that high. I think it would stay down around 5 or 10 per cent at the most.

I would like to bring this question up for discussion and hear what experience the other gentlemen have had.

Mr. Buchanan: May I ask you to describe the particular stripping test you use and to what you apply it?

R. M. Schwegler:
Oregon

We apply it on all aggregates.

Ir. Buchanan: For cover coat, and dense graded mixtures as well?

. M. Schwegler:)regon

Yes, we run it on all aggregates as a matter of record. The stripping test used is a modified Nicholson test. A mixture of bitumen and aggreate is prepared and allowed to cure over night. The mixture and a quantity of water is then placed in a flask and agitated and the amount of stripping observed visually.

.evi Muir: Jtah

In our state we have had materials that are rather absorptive. They frequently carry high percentages of water. After mixing for a day or two, we can frequently break some of the pebbles and see where the oil has penetrated to depths of an eighth of an inch. It seems to me the stripping test—I was going to say—is all wet in some respects, in that very little time is allowed for the oil to become attached to the rock before the water is added and stirring action takes place. We have used some hard, quartzite gravels. Some are clean, lack filler, and are slow setting after they lay them, and I know there is not one out of ten of those gravels that would meet anybody's stripping test, and yet they give perfectly satisfactory results on our roads. We have one road over a wet, swampy country that extends between Ogden and Hooper. There has never been any trouble with stripping on that road. It is low and exposed to water. We have no explanation of why we obtained successful results.

When Julian Powers first developed his test, the first test of the two, the stirring test, on a certain project we had all the material in the road and under contract ready to oil. I checked that up, and it failed miserably under that test. That was eleven years ago. Two years ago I happened to be going past there and obtained a sample of the material from the road and we couldn't make it strip at all; it absolutely wouldn't strip.

I rather think the importance of that test is over emphasized. I think in a lot of cases states are throwing out materials that would give perfectly satisfactory results, judging from our experience in Utah.

Ir. Buchanan: Thank you, Mr. Muir.

. C. Campbell: New Mexico

I think Mr. Muir has hit on the crux of the situation with respect to the stripping test. I don't think it is any more fair to test something for stripping, before it has had time to cure out, than it would be to pour a concrete specimen and try to test it immediately or a short time after pouring. The idea of stripping test is regarded as good, and I think we should give the asphalt material time to cure out, a reasonable time.

Levi Muir: Utah

We depend quite a bit on the swell test. We test material that is smaller than No. 10 mesh sieve, and we have a swell 0.062 inch specified. In certain parts of Utah the materials are inclined to swell. It seems that all available materials are inclined to swell. It so happens that one district is a gravelly area, and we do take some chances. We increase that limit of 0.062 sometimes to 0.100 inch where surface is to be placed on a gravelly base and we are always very careful to seal heavily. We feel in that particular area that there is no danger of water coming from below into the mix, and if we seal it heavily on top we can keep water out of it that way. We have gotten very satisfactory results. It does seem as though those mixtures are inclined to dry out. They are not really as durable as material that does meet the swell test.

H. F. Winterkorn: Missouri

If I may be permitted, I should like to repeat a few things which have been cited in the last two meetings here, things which might be worth repeating.

I think it is known to most of the people who have been working on the stripping problem that the actual stripping in the road depends upon the aggregate, upon the filler, upon the asphalt, and upon the design, which is very important. Therefore, you cannot make a correlation of any stripping test data with the actual behavior on the road, as long as you use different designs. However, you can establish a certain connection for a certain design, or a certain connection for a certain material.

A very important factor concerning the service behavior of a road is the climate. Climatic conditions vary for the different regions, and therefore some material which strips badly under one condition may not strip at all under other conditions.

The stripping test itself fulfills the function of determining the desirable or undesirable properties of the aggregate, although experience must find out whether these desirable or undesirable properties of the aggregate will have a bad effect on the road. The stripping test takes a single property which may or may not be of primary importance in the road, depending upon the design, the climate, and other factors.

In making the stripping test, we have to define, first, the moisture condition under which we coat our aggregates. In rainy weather, the aggregate may not look moist, but it absorbs a thicker moisture film than it absorbs in dry weather. Therefore, we have to submit the aggregate to standardized moisture conditions. The vapor pressure employed must change from one climatic condition to another. The testing engineer must find out how he has to expose his material first to the atmosphere before coating and then how long he has to cure it to get a relationship with the behavior of the material on the road.

In the Research Committee paper I have attempted to show the different factors which influence the actual results of the stripping test. In earlier papers, especially two years ago, I tried to emphasize the points of design. The general problem is similar to that concerning the use of cracked or uncracked asphalt. Assuming that the asphalt tends to be heterogeneous, if the system is dense enough and prevents the air from coming in, the asphalt will be all right. If we have a dense design and a hot mix, we can make excellent pavements with hydrophilic aggregate. In large parts of Europe only, hydrophilic aggregate is available. Neverthless, excellent pavements are constructed by using the proper design.

Another fact pointed out in the paper concerns the case of soil material possessing a great affinity for moisture. If this material is coated at a lower temperature than is attained in the road, 75 per cent of the absorbed water may become free water.

I have tried to treat all these different aspects of the stripping problem. I think going back to some of the older reports and then connecting them with some of our new knowledge will change the point of view of quite a number of people insofar that they will know what one can expect from this test and what one cannot expect, and what has been claimed and what has not been claimed.

We attend meetings and hear somebody talk, and we don't take the time to study the paper. Of course in a speech we cannot present all the important points in a quantitative way but you cannot skip the quantitative aspect of these problems. Therefore, we must study the papers, thoroughly and repeatedly. If we would do that more, I think we would know more about our problems and their possible solutions. I have found I can learn a lot from books written even twenty or thirty years ago on problems we are still discussing here, problems on which there are about as fine solutions as we have now. I want to point to the good old book of Hubbard. I think it was 1909 or 1910 that it came out, and if you will go back and read it again you will find quite a number of answers to problems which we are still discussing. I think we might read a little more.

Mr. Buchanan: I think it might be appropriate to read the summary of the individual papers pertaining to this subject:

"Some form of adhesion tension test has been added by a number of states, especially for aggregates to be used for surface treatment, inverted penetration, and seal cover. A washing or stripping test is used with various modifications. The swell test is still popular, however, for the dense graded mixtures, rather than a test involving agitation."

Are there other comments on this subject? Mr. Flint, I believe you had a question or comment a moment ago.

H. R. Flint:
Idaho

Mr. Buchanan, it wasn't exactly a question, but I would like to say something in defense of so-called cracked oil. I am like the gentleman from Wyoming; I am a little ill at ease in a group of scientists. I feel more at home with the man who is out doing the practical job.

Idaho has many miles of oiled surfaces made with cracked oils. I don't know that we can point to any definite failures where we have provided the proper base to support the oil mat. There is one subject that had a sufficient base under it, and should have been a good job. For some reason or other, that whole mileage of, I believe, about twelve miles pot-holed very badly. We got all the scientific help at our command to determine what caused the failure, but we don't know yet what it was. No one was able to tell us. We followed that failure up with another mat on top of the old one, and there has been practically no maintenance to the surface of that road since. I know, where we have placed oil on some of our early roads that were not sufficiently constructed as far as base is concerned, we have very high maintenance and unsatisfactory surfaces to travel over. That is not the fault of the oil.

I am sure that at the present time Idaho is enjoying far better service from the producers of this oil than they may have in earlier years. All the companies providing us with oil at the present time are having tests made before the oil is shipped and before it has a chance to be placed in the road, which is more satisfactory because in the past we have received the oil, taken the samples and sent them to our laboratory, and long before the tests were made the oil would have been in the highway.

I would just like to say we are not particularly concerned about the importance of the Oliensis test, and I happen to know we are not the only state that feels that way.

Mr. Buchanan: Thank you. Are there other comments? Here is a golden opportunity to get something off your chest.

L. C. Campbell:
New Mexico

In its brief history of road building, New Mexico has had several miles laid down with cracked oil. If the oil wasn't admittedly cracked to begin with, it was later admitted to be cracked, so there is no question about that. For many years New Mexico has kept—at least I think—a very accurate cost accounting of maintenance, and over against that we have the maintenance cost accounting broken down into various items, among which is maintenance on the surface treatment. The figures we have, by and large, on surfaces treated with cracked oil is very hard on a poor Scotchman's heart.

Mr. Buchanan: I think Mr. Flint was referring to mixes, and you are referring primarily to surface treatments. Let's not get those two types mixed in our thinking during this discussion.

Levi Muir:
Utah

Utah, along with Idaho and Wyoming, has many miles of oiled surfaces where cracked oil has been used as binder. I would say that possibly Utah and Wyoming have 3000 miles of cracked oil, principally SC-2 and SC-3. In the days when we had 60 percent road oil specified, we built a lot of road out of this cracked oil. We had a stretch of a mile built in Meadow on U. S. 91, south of Salt Lake about 160 miles, which was put in in 1929, and we didn't seal that road until this year. The fellows didn't know how to use it. It looked black in color compared to the California oil. We blundered into it, and it worked out well. I would say we have possibly 1500 miles of it now. It hasn't all been perfect. We have a lot of oil mats laid on very weak bases. Some of these roads are in poor condition at this time, but the failures cannot be attributed to the binder used.

I think there is one thing in the use of cracked oils that some people may not recognize, especially at first. The cracked oil we use has a high gravity. If you are controlling your mixture by a percentage, naturally you have to correct for that, and it may be possible, in the case Mr. Flint mentioned, that it is just a dry mix with not enough oil for the gravel. That condition could cause potting. Dry mix would ravel, and so on. We have always had to correct for the amount of oil figured on a weight basis, but on a volume basis we have always used the same amount for the cracked oil as we have for an uncracked oil. We oiled in other towns on 91, about a mile or two in each town, with some of this oil, and we also used, I think, two grades of California oil. The jobs have all been uniformly successful. They are all serving right now, and while they are not perfect, we have had no occasion to re-mix any of it.

There is another comment I would like to make on the use of cracked oil. I think it is desirable not to use too heavy grades. I would rather confine our use of cracked oils to SC-1, 1A, 2 and 3. If you go up to SC-6 you are going into a field that is sometimes occupied by paving asphalt, the heavier asphalt, and so on. I think we would be inclined not to want cracked oils for those heavy projects and possibly cutbacks, although we have used a lot of cracked cutbacks.

Mr. Buchanan: Thank you, Mr. Muir. I want to emphasize again this difference in the two comments that were previously made, one referring primarily to mixes, and the other primarily to surface treatments, which I think was clearly and adequately covered by Mr. Benson's report on investigations in Kansas.

H. G. Nevitt:
Missouri

I would like to emphasize the point made by Mr. Muir, that in comparing the so-called cracked oil materials you have to work on a volume basis. If you are dealing with weights rather than film thickness, the positive spot oils are 10 or 12 percent above the negative. Hence, if you are using the same weight percent, you will actually be cutting down the film thickness, and if you are dealing with lean mixes, the 10 and 12 percent reduction in thickness gets down to lean mix that will ravel. I think this has to be kept in mind in interpreting some of the published comparisons between the two materials. They have used the same weight material, even though they bought the material on the volume basis. Equal weight comparisons have been made; hence, if the film thickness has been near the critical point, often that cutting down of the film thickness alone might have been sufficient to make the positive spot material ravel and not make the same showing as the negative spot. The trouble was simply that they didn't compare on a volume basis, as Mr. Muir points out.

Mr. Buchanan: Mr. Cheney tells me our time is up. I want to thank all of you for your whole-hearted cooperation in keeping the discussion going. If there are any points that have not been covered, there will still be an opportunity Thursday morning during the Round Table Discussion, and, of course, you are at liberty to collar anyone with whom you want to talk between now and then.

Papers Prepared

on

Fundamentals of Bituminous Construction

for

The Montana National Bituminous Conference

Durability of Asphaltic Materials

By

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INTRODUCTION

The durability of asphaltic materials is among the most important of a number of factors which combine to form the general problem of serviceability of bituminous surfaces. The factors which make up the general problem of serviceability are closely connected and interdependent to a large degree. The realization of this interdependence has developed rapidly during the past few years.

Many interesting and very valuable papers have been presented before this conference on the subject of durability of asphalts. The writer has endeavored to stress, in this paper, some of the relationships of durability of the asphaltic material paramount to the problem of serviceability of the road surface. Particular attention has therefore been paid to available research data which tends to show the effect of the durability of the asphaltic material on the serviceability of the pavement structure.

The efforts to control the durability of asphalt by tests applied to the material have been briefly reviewed. Application of these tests in today's practice has been secured from a number of highway organizations who returned questionnaires submitted to them for this purpose. Trends in current practice as indicated in the returned questionnaires have also been noted.

OUTLINE OF THE SEARCH FOR GREATER SERVICEABILITY

The search for greater serviceability of asphaltic road surfaces appears to have assumed four major phases. The first of these was early recogized in the advisability of providing a base for the surface course. The second phase has been research into asphalt-aggregate structures to secure maximum durability of the asphaltic material; the third phase, experimental construction employing different types and grades of asphaltic material; and the fourth phase, research into the behavior of asphaltic materials in service and the development of tests to endeavor to predict this behavior.

The First Phase: Bases

With the advancements which have been made in soil science toward an understanding of the functions of soil and mixtures of soils with aggregates, cements and bitumen, the construction of bases for asphalt surface courses has been greatly accelerated. It has been noted that we are today in an era of base construction, since this practice has been widely adopted. The construction of better bases and elimination of one of the most serious sources of trouble in bituminous road construction has incidently placed greater emphasis upon the serviceability of the bituminous surface.

The Second Phase: Design of Asphalt-Aggregate Structures

The design of asphalt-aggregate structures is becoming increasingly attentive to those factors which tend to influence the serviceability of the asphaltic material within the structure. It is today becoming generally recognized that the proper design of asphalt-aggregate structures must include more than the previously considered primary functions of stability, tractive surface and protection from the destructive action of water. Economics of this type of construction has made necessary the added consideration of protection of the asphaltic material from the action of heat, cold and air.

Hubbard and Gollomb[1] have called attention to the dangers of air permeable structures and the rapid changes which may result in the asphaltic material when thus exposed. A design for low air permeability, and sealing were noted as methods of correction of such conditions. Hubbard and Krieger[2] recommend that when the percentage of voids exceeds 6% in all types, the surface should be seal coated. Raschig and Doyle[3] state that paving mixes should be as dense as possible, and that thin open mixes show a high penetration drop (of the asphalt) within a relatively short time. Hveem[4] has called attention to design considerations of pavement structures and enumerated some of these factors as sufficient stability and freedom from ravelling, resistance to weather action, including moisture, heat and cold, oxidation, a non-skid surface to eliminate driving hazards, a tight impermeable surface to protect the subgrade when necessary, or sufficient porosity to permit suspiration of ground water when that is necessary. Economical utilization of local aggregates and ability to form a smooth, dry surface are also included as design considerations. In these and other sources, the consideration of the effect of the aggregate structure on the durability of the asphalt in the design of such structures is noted. The reasons behind this consideration are sound.

In any discussion concerning the durability of asphaltic materials, it must be borne in mind that the asphaltic material itself, whether good or bad, whether from natural lakes in which centuries were required to form the bitumen or from modern refinery units producing asphaltic material in what might be termed a matter of minutes, if exposed to the elements, particularly heat and air, in a high ratio of surface area to mass, tends to become hard, brittle material in a matter of hours or days. In spite of this characteristic of asphalt and its derivatives, some of the poorer materials may give months of service in highway structures while the better materials may remain soft, ductile, adhesive and serviceable over very long periods of time. Records of over 50 years of service, with the asphalt under constant exposure to the elements and protected only by the enclosing structure of aggregate, are not uncommon in American cities. The best asphalt is rendered durable only through the protection given by relatively large masses with minimum surface areas, or by certain aggregate-mineral structures which helpfully enclose and seal the asphalt from attack by exterior destructive forces. It may be noted that occasionally poorer asphalts may be destroyed or rendered unserviceable by time and/or heat alone, apparently regardless of the protection which may be given by any aggregate structure.

The Third Phase: Manipulation of Asphalt Types

The third phase of the search for greater serviceability of asphalt road surfaces, particularly the light or intermediate types of construction employed in the West, has been the manipulation of types of asphaltic materials as distinguished by consistency or curing properties, namely, SC, MC, RC, AC or asphalt cements, and emulsions. Manipulation of type is here inferred to mean the change from one type of material to another, with or without changes in method of construction, as for instance, a change from SC to MC in road mix construction.

From a use viewpoint, the predominating types of materials have varied from heavy penetration asphalts previous to about 1925, to the use of road oils beginning at that time. Road oils were generally favored for mat construction until about the year 1929, when California began experimental work using medium curing cutback asphalts[5]. Definite advantages were soon evident in this type of material. Powers[6] has enumerated five advantageous characteristics of medium curing cutbacks, as follows:

1. It definitely develops a stronger binder than the SC type.
2. It does not lose its workability to any appreciable extent during the mixing period.
3. It will develop sufficient binding strength to successfully lay lower filler content aggregates than can be laid using slow curing oils.
4. It leaves a residual product of rather soft consistency which should contribute to long life.
5. It is less susceptible to displacement by water than are slow curing oils.

In addition to the advantages enumerated by Powers, Krchma[7] has added the following:

1. Tougher surface with better non-skid properties.
2. Less critical proportioning of oil and aggregate.

3. Better specification control of the asphalt present.
4. Higher proportion of a penetration asphalt for a given viscosity or workability.

These and other advantages led to a quick acceptance of this type of material by a number of highway departments. The increase in the use of cutbacks, of which MC formed the major part, is reflected in the increase[8] in cutback consumption of from 130,000 tons per year in 1929, to 446,000 tons in 1931, or an increase of more than three and one-half times in that period. The success and use of the various materials has been reviewed by Krchma[7] and Tremper[9] in the 1938 proceedings of this conference.

Opposed to the advantages and better serviceability derived from MC cutbacks, are some disadvantages which have become evident in the period of its use. Hveem,[5] Russell,[10] and the writer,[11] have noted objectionable behavior in surfaces constructed with MC cutbacks, in that such surfaces tend to rapidly acquire hardness and brittleness, leading to extensive cracking. Such conditions require frequent sealing of the surface, increasing the cost of this type of construction and decreasing or nullifying to a certain degree the advantages noted for the material. That this brittleness is due, in part at least, to the character of the asphalt is indicated in the results of extraction and recovery tests of the asphalt in such surfaces.

Smith[9] of Oregon has noted another disadvantage of MC cutbacks in the apparent inability to secure complete volatilization of the solvent in this type of material during the construction period. The State of Oregon has therefore used RC materials in lieu of the MC types. Another disadvantage of MC types which has been noted, is the difficulty of reworking such surfaces when such work is necessary.

These disadvantages, with possibly others in the form of slightly higher cost in some areas, have led to changes in the use of the material by some organizations. Several states and other organizations, as noted by Krchma,[7] have revised specifications for MC materials to require a soft, 200 to 300 penetration distillation residue asphalt, correcting former specifications, which to a certain extent prohibited the use of residues over approximately 150 penetration. California has secured an effective compromise between MC and SC type materials in their ROMC types, which are essentially heavy SC road oils cutback with kerosene type solvent. Hveem[5] states that this material is in no respect inferior to the standard MC grade and has the distinct advantage of having less tendency to become brittle and is workable under a wider range of conditions.

Other organizations have changed to materials of asphalt cement type or to RC materials, involving changes in methods of construction, or have reverted either wholly or in part to SC type oils. Some organizations during the period of use of MC materials, have consistently favored the SC types. The efforts to retain the advantages and overcome the disadvantages of the MC types are noteworthy, however.

Changes of type of asphaltic material to secure better serviceability have taken place in other forms of construction besides bituminous mat.

In the field of surface treatments and armor coats, there has been a distinct trend toward heavy MC, RC, and soft asphalt cements. The heavier materials tend to give better serviceability through greater binding properties, and also possibly through greater durability of the asphalt by the assumption of massive films in the aggregate structure, an important form of protection of the asphalt in such highly exposed construction.

In the field of asphalt pavements of the hot-mix type, much data has been accruing on the behavior of asphalt in service, and the conclusions reached on this data have influenced changes in asphalt specifications. Softer, or high penetration asphalts, are now recommended, asphalts of 100 penetration or higher being used in construction formerly employing 40-60 penetration asphalts. The apparently satisfactory behavior of the softer asphalts has been noted by Hveem[4] and others.

In still another field of construction using asphaltic material, that of bituminous soil stabilization, no definite data is yet available as to the comparative efficiency of the various types. All of the common types of asphaltic material, SC, MC, RC, and asphalt emulsions, together with tars, have apparently been successfully used. Until special cases arrive, or performance and durability difficulties arise, the continued use of the various types may be expected.

The Fourth Phase: Research on Pavements and Development of Tests

The fourth phase of the search for greater serviceability of bituminous surfaces, the research into the behavior of asphaltic materials in old road surfaces and the development of tests to predict that behavior, is one which has claimed much attention and which today may be considered as the major phase of the attack on the problem of serviceability. This phase consists of two parts, the work which has been done in determining the behavior of the asphaltic material in service, and the work performed in developing tests for the prediction of that behavior. The two parts are bound very closely and are here considered as comprising one phase.

The research into the behavior of the asphaltic materials in the road surface has been accomplished chiefly by two methods, the first being the extraction and recovery of asphalt from the surface, followed by tests on the recovered asphalt; the second, the testing by physical means of speciment (cut from old pavements) to determine the physical characteristics of the asphalt-aggregate structure. From such characteristics, it may be possible to deduce the character of the asphalt binder. These two methods are frequently used together and when so used, furnish much interesting data.

EXTRACTION-RECOVERY METHOD OF RESEARCH

The first method of investigation, the extraction, recovery and testing of the asphalt in pavement surfaces is not new. Dow[13], in 1903, described a method of recovery of asphalt employing CS_2 as a solvent. Improvements on this method were proposed by Bateman and Delp[14]

(1927) and by Bateman and Lehmann[15] (1929), in which partial vacuums with a limited maximum temperature of distillation facilitated the removal of the solvent (CS_2) from the asphalt while tending to prevent excessive changes in the asphalt due to high distillation temperatures. Greutert[16] in 1932 described improvements over previous methods of recovery by using CO_2 gas in the preliminary stages of distillation under a low partial vacuum. The maximum temperature of Greutert's recovery method was limited to approximately 180° F. above the softening point of the asphalt being recovered. Greutert utilized benzol as a solvent. Abson[17], in 1933, apparently overcame many of the objections of the previous methods by the use of benzol as a solvent and by controlled rates of flow of CO_2 gas during distillation, and by limiting the time and temperature employed during recovery. Check tests employing this method have indicated a high degree of accuracy.

A method proposed by Bussow[18] in which benzol is used as a solvent but CO_2 gas is not used in the recovery has been used by some investigators. This method while not possessing the accuracy ascribed to the Abson method, has some advantages of simplicity and economy.

PHYSICAL TESTING OF PAVEMENT STRUCTURES

The second method of asphalt behavior investigation, the testing of specimens cut from pavement surfaces, has been used by several investigators, most prominent of which are Rader[19] [20], and Raschig and Doyle[3] [21]. Rader, in investigating the behavior of asphalt-aggregate structures at low temperatures using the structure as a simple beam, determined, among other characteristics, Moduli of Elasticity and Moduli of Rupture of the structures.

Raschig and Doyle continued, in part, the investigation of Rader, using different methods of test. Raschig and Doyle used both new specimens and specimens cut from pavements as cantilever beams and determined Moduli of Elasticity and Rupture at temperatures of –5° F., 25° F., and 38° F. on the new specimens ,and at a temperature of 38° F. on the pavement sections.

Vokac,[12] [25] differing from the investigators who performed tests directly upon pavement specimens, disintegrated and then reconstructed by heat and pressure, structures of the pavement asphalt and aggregate. Correlation to pavement conditions in the reconstructed structures were secured largely by the characteristics of compressive strength, elastic limit and modulus of permanent deformation.

RESEARCH BY OBSERVATION

A third form of research which may be mentioned since it is adapted to investigations of thin surfaces such as seals or light armor coats, where samples of sufficient size for laboratory examination are difficult to secure, is the observation method. Data is secured by making observations on the condition of the asphalt and the surface, and from data secured from a number of materials used under similar conditions, much interesting and valuable information may be secured. A brief summary of work of this character performed by the writer is included to illustrate the point.

DISCUSSION OF RESEARCH METHODS

Certain advantages and disadvantages are present in the two methods of research previously described and affect the interpretation of data secured by these methods. The extraction and recovery of asphalt has several aparent disadvantages, chief of which is the question as to whether or not the recovered asphalts have the same characteristics as the material before extraction. There is indication that this questioning is justified in the fact that thixotropic reactions occur in asphalts which are reversible by kneading, manipulation, heating, or by the action of solvents. Asphalt from a cracked surface may show a penetration, for example, of 60 when recovered from solution, yet, due to thixotropic hardening, this material may have been at 30 or less penetration when the cracking occurred. Work by the writer[22] has indicated that some reactions occur which are reversible by the addition of solvents, that unasphalt-like substances may be converted back into material having asphalt characteristics by addition of CS_2, CCl_4, or benzol. On certain materials, this reaction may produce misleading data. Finally, there is some question as to the proper interpretation of data secured by this method with respect to the durability of the asphaltic material.

The advantage of the asphalt recovery method is that a much clearer insight into the actual behavior of asphaltic material in service has been secured. Actual measurements may be made on the asphalt to determine the comparative rate of change taking place in the material in service and in research. In the hands of the investigator, the extraction-recovery method is an extremely valuable tool. An outstanding example of the use of this test method is the work of Brannan[23] of Ohio, who determined that very large drops in penetration were taking place in the mixing procedure of hot-mix pavement construction, the drops frequently being equivalent to one or more years of service. The correction of the difficulty and constant checks by means of this method have resulted in a greater life anticipation of Ohio's hot-mix asphalt pavements.

The disadvantages of the second method of investigation, that of the physical testing of pavement specimens, is considered as due mainly to the complexity of the factors entering into the characteristics measured. Oliensis[24] has noted that this method introduces a multiplicity of confusing factors due to variations in aggregate, differences in interfacial tensions, problems of mixing and compaction and many others. Vokac[25] has noted that in his experience the securing of undisturbed specimens from the pavement surface is very difficult. The testing of surfaces using SC type materials where very little cohesion exists also presents serious disadvantages. Where the structure must be reconstructed by recompaction or otherwise disturbing the existing structure of the material as it exists in the pavement, the method possibly loses one of its greatest advantages, the determination of the actual existing physical properties of the pavement.

The advantage of the method where undisturbed specimens can be secured, appears to lie in the ability of the test to provide a summation of a number of properties which may be expressed in one or two simple units. The use of these units may prove of as much value, or perhaps more value, than the results obtained from extraction-recovery tests in certain types of construction in ascertaining the meaning of "durability of asphalt."

DEFINITION OF DURABILITY OF ASPHALT

The meaning of durability with respect to asphaltic materials is not well defined. "Durability" implies the ability to resist change, but as noted by Hubbard[26], no asphaltic material so far as is known, is capable of completely resisting change. Changes begin at the moment of manufacture, they continue through the mixing operations, accelerated by air, heat and other factors, retarded by protective aggregates and cold, until eventually the asphalt no longer fulfills the requirements placed upon it by design and some type of failure becomes evident. The most commonly accepted evidence of distress or failure is cracking of the surface. The characteristics producing failure would be expected to vary with the asphaltic material, the structure of the aggregate, mean, maximum or minimum temperatures and use. The point to be made is that durability does not refer to one particular state, but, since asphalt is constantly changing, durability refers to the period of time required by the asphalt to change from a condition in which it meets all the requirements placed upon it by design, to the condition where it will no longer perform these functions satisfactorily, assuming that the other factors which enter into the evidences of failure have been properly evaluated, and that the material did meet design requirements at the time its actual service began. Another consideration enters, in that the acceptance of cracking as an evidence of failure signifies only the end of one period of durability, which might be called the period of "optimum durability." The surface may remain in service, assisted by sealing or proper maintenance for a considerably longer period until complete failure ensued or new construction was warranted. The time required for that stage to be reached might be called "total durability" of the asphalt. Investigators into the durability of asphalt have apparently accepted by common assent that period noted as "optimum durability," particularly referring to sheet asphalt pavements. With these definitions as a base of reasoning, the data secured from research may be better evaluated.

MEASUREMENT OF "OPTIMUM DURABILITY"

Extraction-Recovery Method

An effort to evaluate the data secured from extraction-recovery tests with respect to the cracking of pavements has been made by Hubbard and Gollomb[1]. Their summarization of data is as follows:

1. When the penetration of the asphalt cement falls below 20, serious cracking may be expected to develop in the pavement.
2. Some cracking in the pavement may be expected to develop because of the hardness of the asphalt when its penetration drops below 30.

3. High resistance to cracking may be expected from a well designed and properly compressed pavement when the penetration of the asphalt is well above 30.

It is noted that the limits which have been stated apply chiefly to northern and middle belt cities.

Miller, Hayden, and Vokac[68], summarizing the penetration characteristics of bitumen from a large number of pavements classified as cracking, sound and shoving have indicated the following.

Classification	Bitumen	Penetration
Cracking	Trinidad	28
Sound	Trinidad	43
Shoving	Trinidad	55
Cracking	Oil	18
Sound	Oil	29
Shoving	Oil	39

Powers[27] has noted that pavements in which the penetration of the asphalt is above ten, appear to be free from cracking, this data being from a southern area.

Although the data presented by Hubbard and Gollomb[1], Miller, Hayden and Vokac[68], Powers[27], and others indicates that the property of penetration is the factor for determining the behavior of the surface with respect to cracking, other properties are also indicated as affecting the behavior of the surface. Steinbaugh and Brown[28] have indicated that the ductility value of recovered asphalts from good and cracking pavements showed, for the cracking pavements, ductility values which average less than one-third those of good pavements, and that the differences in this property (ductility) exceeded the differences in penetration. Steinbaugh and Brown also state that badly cracked pavements had an average ductility of 10 cm., while good pavements had an average of 32 cm.

McKesson[29] has presented data showing rapid drops in ductility as a result of oxidation of the asphalt in the laboratory, noting that the ductility values frequently varied more rapidly than penetration values. McKesson, in the data supplied by Proctor and Shattuck[29], has indicated that the value of ductility may be of much importance in determining the behavior of the surface, although strict correlation is not present.

Raschig and Doyle[3] [21] note that in their tests on recovered asphalt from good and cracked pavements, the condition of the pavements in general were poorest where the ductility and penetration were lowest.

Miller, Hayden, and Vokac[68] has tabulated, without comments, the average ductility characteristics of a number of recovered bitumens, as follows:

Classification	Bitumen	Ductility at 77° F.
Cracking	Trinidad	76
Sound	Trinidad	139
Shoving	Trinidad	146
Cracking	Oil Asphalt	6
Sound	Oil Asphalt	40
Shoving	Oil Asphalt	70

Physical Testing of Structures

The physical testing of aggregate-asphalt structures from old pavements has given data, largely through the work of Rader[19] [20], Raschig and Doyle[3] [21], and Vokac[12] [25], which indicates that correlation to cracking of pavement surfaces and the durability of asphalt may be determined by this method also.

The data of Rader[19], who used beams of the structures at –70° F. to secure the data, indicates that the following conditions existed in cracking and non-cracking pavements, the data here given being the average of four or more specimens:

Pavement Condition	Pen. 77°	Modulus Rupture –70°	Modulus of Elasticity –70°
Good	56	1600	479,000
Bad	42	1175	624,000

Rader[19] obtained his data from two streets, one of which was badly cracked, and one which was in good condition.

Raschig and Doyle[3] also submit data, the average of a number of tests on pavement structures:

Pavement Condition	Pen. 77°	Modulus Rupture 25°	Modulus-of-Elasticity 25°
Good	37	1300	113,300
Bad	20	955	332,200

According to Rader[19] [20], and Raschig and Doyle[3] [21], the modulus of elasticity is a measure of brittleness, or a flexibility factor. The above data then indicates that by means of this test, it may be possible to indicate the approach to a critical cracking condition of the pavement, or the end of the period of optimum durability if the modulus of elasticity of the complete structure falls within the following limits:

At –70° F.	Between 480,000 and 625,000
At 25° F.	Between 100,000 and 300,000

Vokac[12] [25], using reconstructed asphalt-aggregate structures from old pavements in contrast to the undisturbed specimens used by Rader, and Raschig and Doyle, obtained data from a number of tests indicating that three physical characteristics tend to distinguish between shoving, sound and cracking pavements. A brief summary of this data in which only 80% of each group of tests has been used by Vokac to prevent overlapping of the groups, is as follows:

Characteristic	Lower-Limit Cracking Group	Upper and Lower Limit Sound Group	Upper Limit Shoving Group
Compressive Strength	1000	1225-575	600
Elastic Limit	850	1025-525	525
Modulus of Permanent Deformation	4750	5750-1750	2000

This data indicates that with this method of test, a critical cracking condition of the pavement is approached when the compressive strength of the asphalt-aggregate structure reaches or exceeds 1000-1225 pounds per square inch, when the elastic limit is at or greater than 850 to 1025 pounds per square inch and the modulus of permanent deformation reaches or exceeds 4750 to 5750.

Vokac, in using shoving as a serviceability factor, has indicated not only the limits tending to produce cracking, but also the limits in the other direction toward flexibility and softness which may be approached without attaining instability and poor serviceability from these factors.

CONCLUSIONS

The work performed by means of the extraction-recovery method of determining the ''Optimum Durability'' of asphaltic materials establishes certain values of physical properties which tend to indicate the point where the behavior of the material changes radically and a period of optimum durability of the asphalt is ended. The physical properties of the asphalt used to indicate this condition may be penetration, ductility, or both. This method tends to stress highly the characteristics of the asphaltic material.

The research of asphalt-aggregate structures also indicates that certain values of various physical characteristics of such structures, denote the end of optimum durability periods of the asphaltic materials. The structure functions which are most commonly used for such determinations include Moduli of Rupture and Elasticity at various temperatures, Compressive Strength, Elastic Limit, and Modulus of Permanent Deformation

The research on asphalt-aggregate structures tends to stress the relationship of the physical characteristics of the structures to the tendency of such structures to become brittle and crack and in so doing indicates and determines the comparative importance of the character of the asphalt and the character of the aggregate structure as each operates to cause or prevent the cracking condition.

APPLICATION OF SPECIAL RESEARCH TO GENERAL TYPES

Much of the discussion of the methods and the results of research performed on asphaltic materials refer to pavements of the hot-mix or sheet asphalt type. The results of such work may be considered by some as not applying to surfaces constructed with materials of SC, MC, RC or emulsion types. With respect to work performed on materials of MC, RC, and emulsions, it has been noted that the asphaltic material in these surfaces tends to behave much the same as asphalt in the higher type surfaces, especially where the structures are similar.

Carney and Grant[30] indicate results from extraction of surfaces employing RC material although no comment is made on the condition of the surface. McKesson[29] has presented data on the behavior of emulsions which indicate the applicability of our tests to this type of material. The writer[11] has previously commented on behavior of some MC materials in service, in which cracking of surfaces constructed with this type of material apparently has been reflected in low penetrations and ductilities of the asphalt recovered from these surfaces.

Correlation of the behavior of SC materials and the surfaces constructed with this material presents more difficulty. Although such material frequently reaches consistencies in which penetration, ductility and other normal functions may be measured, the greater part of its useful life is spent in a condition in which few of our favorite tests can be applied, yet some of the material possesses, apparently, excellent durability. Since this type of material tends to exercise its functions as a binder while in combination with very fine mineral matter, the function of the matrix thus formed may be of more importance than physical properties of the asphalt itself. Physical tests on materials and mixtures of this type have a promising future. The work of Bollen[31] is cited as a step in this direction.

Intermediate and light type asphaltic surfaces are perhaps more concerned with retention of flexibility and durability of asphalt, because of greater stress laid on this function of the surface by relatively poorer bases, than are higher types of construction. The durability of the asphalt is economically no less important in intermediate and light types since the greater mileage of the latter probably more than compensates for the greater unit cost of the higher types. Research on the higher types of construction should be, and our review indicates that it has been, followed closely in the field of the lower type surfaces.

TESTS APPLIED TO ASPHALT

From a review of the methods which may be used to determine the durability of asphalt and the progress which has been made to establish the boundaries of durability, we turn to the asphaltic material and tests which pertain to the material, controlling, or endeavoring to control the immediate and the future behavior of the asphalt.

Tests may be divided into two groups, one of which pertains to the immediate characteristics of the material, making possible the incorporation of the asphalt into the desired structure safely and economically and with sufficient strength to properly bind the parts into a serviceable whole. The second pertains to the behavior of the material after service has begun, to insure, if possible, the continued satisfactory performance of the asphalt in its functions as a part of the structure.

Tests of the first group include, among others, the following:

1. Penetration.
2. Viscosity.
3. Float.
4. Softening Point.
5. Distillation.
6. Flash.
7. Solubility in CS_2.
8. Specific Gravity.

Tests of the second group, those which endeavor to predict future behavior, are more numerous. Some of the tests in use or of recent origin are as follows:

1. Oliensis Spot Test.
2. Pfeiffer-van Doormaal Penetration Index.
3. Fluidity Factor.

4. Viscosity Index.
5. Sulphur Content.
6. Ductility (normal).
7. Low penetration ductility.
8. Low temperature ductility.
9. Temperature susceptibility.
10. Solubility in CCl_4.
11. Insolubility in 86° Naphtha.
12. Wax content.
13. Xylene Equivalent.
14. Loss on heating.
15. Benson Film Test.
16. Weatherometer.
17. Source Requirement.
18. Chemical Constituent Analysis.
19. Nicholson Oxidation Test.
20. Raschig-Doyle Oxidation Test.

CLASSIFICATION OF "PREDICTING TESTS"

The tests which are here enumerated have been considered as affecting the future behavior of the asphaltic material. Some of these tests may frequently not be considered as indicators of future behavior, but research or usage has so established them.

The tests for predicting future behavior may be classified into several groups. The first group is determined by the susceptibility of the asphaltic material, or physical changes in one or more properteis with change in temperature. The following tests may be classified directly as susceptibility tests, and frequently indirectly as identification tests:

1. Pfeiffer-van Doormaal Penetration Index.
2. Fluidity Factor.
3. Viscosity Index.
4. Low Temperature Ductility.
5. Temperature Susceptibility.

A second group is composed of tests which depend on chemical relationships of solubility and an interpretation of the meaning of certain reactions resulting from the action of various solvents. Tests of this type are:

1. Solubility in CCl_4.
2. Oliensis Spot Test.
3. Xylene Equivalent.
4. Chemical Constituent Analysis.
5. Insolubility in 86° Naphtha.

The third group is composed of those tests in which the asphalt is subjected to conditions simulating in some degree the action of the elements and by which greatly accelerated reactions are obtained. This data is then interpreted to predict behavior in aging. The following are acceleration tests:

1. Loss on Heating.
2. Benson Film Test.

3. Weatherometer.
4. Nicholson Oxidation Test.
5. Raschig and Doyle Oxidation Test.

A fourth group is composed of physical tests, the exact functions of which are often in doubt, but which have been accepted, the first generally, as indicating desirable characteristics in both immediate and future behavior. These tests are:

1. Ductility (normal).
2. Low Penetration Ductility.

Two tests, one of which is considered by some authorities as favoring good future behavior, the other detrimental, are:

1. Sulphur Content.
2. Wax Content.

The first of these, the sulphur requirement, is frequently considered as a source, or identity, test.

The remaining test, the definite requirement of source, is used where competitive bidding between different sources is not necessary.

DISCUSSION OF "PREDICTING" TESTS

Susceptibility Tests

The Pfeiffer-van Doormaal Penetration Index (J. Ph. Pheiffer and P. M. van Doormaal, 1938[32]) groups asphalts into three more or less arbitrarily chosen classes on the strength of their differences in temperature susceptibility as shown by the penetration at certain temperatures and the softening point by the Ball and Ring Method. This method has apparently not been used in selection of paving asphalts, probably due to lack of behavior information on the classes of material so differentiated.

The Fluidity Factor Test (Joseph Zapata, 1935[33]) was developed to classify asphalts according to the source of the crude from which they were produced. The test has been in use in Wisconsin and several other states. Carney and Grant[30] have presented interesting discussion on the operation of this test.

The Viscosity Index Test (E. W. Dean and G. H. B. Davis, 1929[34]) was originally developed to differentiate between lubricating oils. This test formed a base for the development of the Fluidity Factor by Zapata and has recently been adopted as an identification test by at least one western state. As an identification test, it is apparently performing satisfactorily.

Low Temperature Ductility (Reference—Walter H. Flood[35], H. Abraham[36] and others[29]) is noted by Abraham to yield, when used with ductilities obtained at higher temperatures, much information regarding the origin and characteristics of the asphaltic material. The test has recently been considered of value as an indication of the tendency of pavements to crack in extremely low temperatures.

Temperature Susceptibility (Reference—Thompson[37], Abraham[36], Mack[38], and others [39] [40]) like the other susceptibility tests, is of value chiefly as a means of identification of material.

Chemical and Solubility Tests

Solubility in Carbon Tetrachloride (Clifford Richardson and C. N. Forrest, 1905[41]) was primarily proposed for the determination of carbenes in CS_2 soluble material, but it is frequently used today to determine the total material insoluble in CCl_4, which includes carboids, ash and carbenes. Kirschbraun[42] has noted that when the carbenes exceed 0.5% in petroleum asphalts, their quality is to be regarded as questionable.

The Oliensis Spot Test (G. L. Oliensis, 1933[43]) was developed to determine qualitatively the degree of physical heterogeneity in asphalts. Due largely to conditions existing in the production and use of asphaltic materials at the time of its introduction, this test has been very widely adopted and is today the subject of much controversy regarding the value of the test and the two classes of material which it defines.

Xylene Equivalent (G. L. Oliensis, 1936[44]) is a method of determining, quantitatively, the homogeneity of asphalt. This test is being used by several departments as noted in comments by Powers and O'Hara[45] and others. It is usually used in conjunction with the Oliensis test.

Chemical Constituent Analysis[36] [46] [47] is commonly based on the determination of the asphaltenes, resins and oily constituents in the asphalt, although the determination of free asphaltous acids and asphaltous acid anhydrides is sometimes performed, Zapata[48] and Thurston[49] have noted some definite relationships of the proportioning of some constituents to the weathering properties of the asphalt. Hveem[50] has recently commented on the effect of these constituents on the durability of the material.

Insolubility in 86° A.P.I. Naphtha (References—Abraham[36], Baskin[51]) is a method of determining the asphaltene content of asphalt. Although asphaltenes are responsible for the plastic properties of asphalt (Saal and Koens[52]), no definite relationship appears to exist between the amount of this material and the behavior of the asphalt (Baskin). The test may be used for identification purposes.

Acceleration or Weathering Tests

Loss on Heating (References—Hubbard[53], Law[54], Zapata[55]) is widely used in specifications for asphalt cements and SC type oils to determine volatilization. Some investigators have noted correlation between loss on heating and hardening of the asphalt in service with time (Zapata[55]).

The Benson Film Test (J. R. Benson, 1937[22]) is a method by which very thin translucent films of bituminous material are exposed to agencies of air, heat and/or radiations to produce changes in the material which are observed microscopically. The test has been used experimentally only and its value has not yet been established.

The Weatherometer Test and Similar Tests (References—Lewis and Hillman, 1935[56], Zapata, 1937[48], Thurston, 1937[49], Streiter, 1930[57] [58]) employ either actual exposure to the elements of weather, or use accelerated weathering utilizing visible and ultra-violet radiations, heat, and water. The application of this test method to asphaltic material used for road construction has been slow due to the difficulty of the time required, test methods for the soft and liquid asphaltic materials and lack of correlation to actual serviceability.

The Nicholson Oxidation Test (Victor Nicholson, 1937[59]) has been proposed by Nicholson, employing elevated temperatures and blowing of the sample with air, to determine the hardening characteristics of the asphalt, and from these characteristics to predict the behavior of the material in service. The test is proposed, but has not been accepted, insofar as is known, for specification purposes. The test may be of particular value to control mixing operations of hot-mix plants.

The Raschig and Doyle Oxidation Test (F. L. Raschig and P. C. Doyle, 1937[60]) employs methods similar to those employed by Nicholson, with the drop in penetration due to oxidation being measured. This test, like that of Nicholson, may be of particular value in controlling mixing operations of hot-mix plants. These two latter tests are of interest since they represent a trend in tests and are, in part, a result of increased interest in mixing control.

Physical Tests

Normal Ductility (A. W. Dow, 1903[13 61 62]) is considered as the measurement of ductility at normal temperatures and consistency, to differentiate normal from low temperature and low penetration ductilities, the latter being special fields.

Normal ductility may frequently be considered as non-predicting test since the properties measured are considered of immediate importance to the asphalt-aggregate structure. The dividing line is not sharp, however, and the use of ductility measurements on materials reduced to, or in penetration by heating or cook down, such as very soft AC and SC type oils, can be considered as no less than predicting tests, since there is no certainty that the material would reduce in service to the same material secured by reduction. Since this applies to distillation also, the only use of ductility where the above reasoning does not apply is to AC of paving grades below about 150 penetration, and even here the tendency is to evaluate the results in terms of future behavior. The ductility test is therefore considered as almost wholly predictive in character.

Low Penetration Ductility (H. Allen and W. E. Gibson, 1935[63]) is considered as the reduction of the asphalt to a low penetration, usually from 50 to 15 at 77°, 100 g., 5 sec., and the subsequent measurement of ductility at 77°.

This method, accepted by several departments, is believed to be used mainly in Kansas in testing positive Oliensis materials of SC, MC, and RC types. The test in this state is specified as a result of research on the properties of extracted asphalts and behavior of the road surfaces. The actual property which is being measured, like the normal ductility, is controversial. Likewise, the effect of the measurement as a predictive test is doubtful. Ductility is essentially a measure of cohesion, but the results of this measurement are frequently considered to indicate other properties.

Sulphur and Wax Tests

The Sulphur Requirement (References—Nicholson[64], Mack[35], McKesson[29], Bencowitz and Boe[65]) probably originated as an identity or source determining test. It is still used for this purpose by some departments. The value of the test is problematical since sulphur may be added to asphalt originally deficient in this material and many known asphalts of good durability will not meet sulphur requirements. It has been noted (McKesson[29]) that sulphur tends to accelerate hardening and loss of ductility, conditions which it is presumed to prevent. This test, like many others, is dictated largely by experience.

The Limitation of Wax Content (References—Abraham[36], McKesson[29], Spielmann and Hughes[66]) were specified, is due to the effect of wax in reducing ductility and adhesion and increasing susceptibility to temperature changes. The test is not generally specified, due to inaccurate methods of determination and the ability to control the material by means of other tests, notably by ductility.

Definite Requirement of Source

The use of a Definite Requirement of Source in specifications is another requirement dictated largely by experience. Its use is not permissable on construction controlled by certain agencies, but it is frequently used by departments where open specifications are not required. Different opinions prevail as to preferred sources, but since such requirements have been in use continually by a number of groups, its use is evidently satisfactory.

THE MECHANISM OF DECREASING SERVICEABILITY

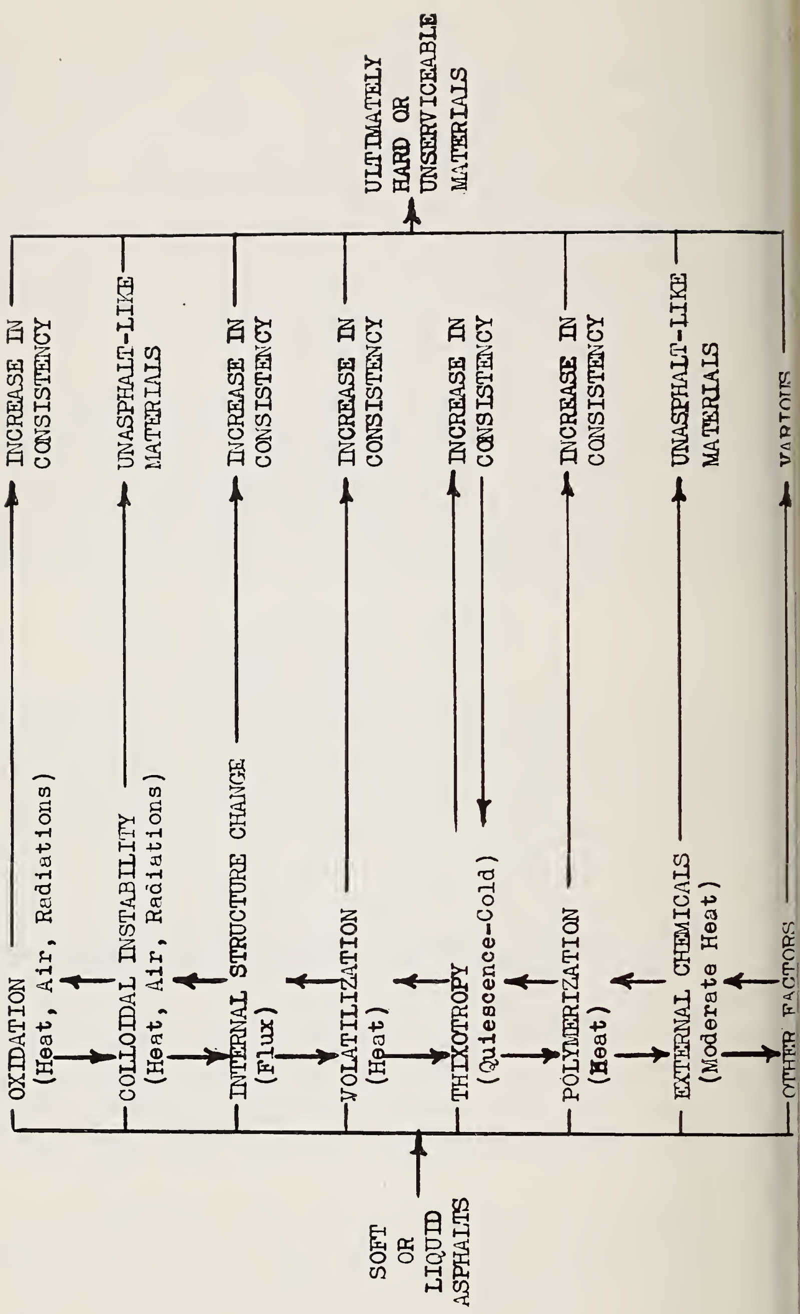
The decrease in serviceability of the asphaltic material is due to a number of very complex asphalt factors, accelerated or retarded by such external forces as air, light, heat, and stress. Some of the common reactions which occur in asphaltic materials to cause them to harden, decompose or otherwise lose their serviceability have been tabulated in figure 1.

The common factors which have been considered here are Oxidation, Colloidal Instability, Internal Structure Change, Volatilization, Thixotropy, Polymerization, External Chemicals and Other Factors. The results due to oxidation have been well discussed in the literature. Colloidal Instability may refer either to theory connected with the Oliensis Test, or with comments which have been made regarding the reactions in translucent films in the writer's work on these films[22]. Volatilization is the most simple of all the reactions. Thixotropy is the reversible increase in consistency of asphaltic materials, frequently of importance in the cracking of ''dead end'' street pavements. Polmerization, or molecular condensation, may occur in chemically unstable material. Some asphaltic materials are susceptible to injury by chemicals which may be present in aggregates or soil used with or adjacent to these materials. Internal structure change has particular significance to the behavior of cut-backs of the MC and RC type and may be closely allied to polymerization. The excessive hardening of some asphalts of this type appears to be due to the action of the

Figure 1

REACTIONS BY WHICH ASPHALT MAY DECREASE IN SERVICEABILITY

Accelerating Factors are Noted Under the Reaction


SOFT OR LIQUID ASPHALTS
OXIDATION (Heat, Air, Radiations)
INCREASE IN CONSISTENCY
COLLOIDAL INSTABILITY (Heat, Air, Radiations)
UNASPHALT-LIKE MATERIALS
INTERNAL STRUCTURE CHANGE (Flux)
INCREASE IN CONSISTENCY
VOLATILIZATION (Heat)
INCREASE IN CONSISTENCY
THIXOTROPY (Quiescence-Cold)
INCREASE IN CONSISTENCY
POLYMERIZATION (Heat)
INCREASE IN CONSISTENCY
EXTERNAL CHEMICALS (Moderate Heat)
UNASPHALT-LIKE MATERIALS
OTHER FACTORS
VARIOUS
ULTIMATELY HARD OR UNSERVICEABLE MATERIALS

flux on the base asphalt. This has been noted and indicated in an investigation by Hveem[5], and by the work of Lewis and Hillman[56].

The complexity of the hardening or decomposing reactions may be estimated when it is considered that all of these factors may be, and probably are to varying degrees, functioning to produce changes in the asphalt. In view of the large differences which may exist in the method of operation of these factors, the reasons for the failure of any one test which may be applied to asphaltic materials to successfully control or predict the behavior of the material becomes apparent. It would appear that such a control, when achieved, will be the result of more than one test, and probably the imposition of more than one condition of use. No ne test which has as yet been proposed, covers or pretends to cover all f the factors which influence the service behavior of asphalt.

TABLE 1

EXAMPLES OF THIXOTROPIC REACTIONS IN SAMPLES OF RECOVERED ASPHALTS

Original Materials—MC Cutbacks

Oliensis Spot Test	Pen. Before Storgae 77°	Undisturbed Penetration 77°	Reheated Penetration 77°	Period of Sealed Storage Months*	Org. S.P. B. & R.	Reheated S.P. B. & R.
P	37	26	32	17	128	128
P	45	28	35	17	120	126
P	14	10	13	17	158	158
P	32	14	25	17	142	144
N	70	36	56	12	122	137
N	67	34	56	12	123	140
N	60	30	57	11	128	134
P	33	14	32	11	142	147
P	25	12	20	11	133	139
N	72	32	62	11	125	128
N	26	14	26	11	158	160
N	76	45	71	11	128	140
P	15	8	15	10	163	169
P	27	15	24	11	135	140
P	25	12	21	11	132	133

	Positive	Negative
Average Total Drop in Penetration	12.7 Pen.	30.0 Pen.
Average Gain in Peneteration on Reheating	8.7 Pen.	22.8 Pen.
Per cent Permanent Loss, Based on Initial Penetration	14.2%	11.6%
Per cent Thixotropic Change, Based on Initial Pen.	30.8%	37.0%

Negative Oliensis Asphalts, in these materials, suffered comparatively arge drops in penetration and almost correspondingly large gains on reheating. The permanent losses in positive materials were larger. Higher average penetration in negative materials noted above probably affected the differences between the positive and negative material.

Thixotropic changes may cause evidences of failure in pavement surfaces if the penetration (and/or ductility) drop into critical ranges. Such reactions tend to explain "dead end" street pavement cracking.

DISCUSSION OF QUESTIONNAIRE

A questionnaire was submitted to each of a number of state, provincial, county and city highway departments, from which 41 replies were received. The questionnaire requested information regarding type of construction and the types of bituminous material used in that construction A tabulation of eighteen tests was included, each test to be indicated i preferred or used in connection with the testing of the bituminous ma erial.

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*The rate of the development of thixotropic changes is indicated in the percentage of penetration drops, based on the total drop occurring in five months on positive and negative MC distillation residues, as follows: Positive at 100 hours, 40 per cent; at 400 hours, 60 per cent; at 74 days, 77 per cent; total drop, five months, from 70 to 40. Negative at 100 hours, 31 per cent; at 400 hours, 52 per cent; at 74 days, 81 per cent; total drop, five months, from 87 to 47. A portion of these drops, not determined, is permanent change.

Three questions were also included to secure information regardin (1) trends in construction, such as road mix to plant mix, open grade to dense graded aggregates, heavy mats to surface treatments or vic versa; (2) trends from one type of bituminous material to another; an (3) a report as to the existence of research on the durability of bituminous material.

Comments regarding the use of the Oliensis test were invited.

ANALYSIS OF REPLIES

Construction and Types of Materials

Construction types have been classified as High Type, Intermediate Plant Mix, Intermediate-Road Mix, Surface Treatments and Soil Stabilization. These types of construction and the materials used as noted in replies to the questionnaire are graphically analyzed in Figure 2.

High Type pavements, although tending to higher penetration, still favor, it is indicated, asphalt cements of the range 30-100 penetration. Comments indicate that the actual ranges vary from approximately 50 to 100 penetration. AC of 100-250 penetration is second in asphalt preference.

Intermediate-Plant Mix construction employs MC cutback asphalt predominately, followed in order of preference by SC road oils and RC cutbacks (equal preference), AC of 30-100 penetration, AE (asphalt emulsion), tars and AC of 100-250 penetration.

Intermediate-Road Mix construction expresses somewhat different preferences with MC predominate, but followed closely by SC and by RC materials. Tars, with nine replies indicating their use for this type of construction, rate a good fourth.

Surface Treatments have a noted preference for RC cutbacks, with twenty-seven departments noting use of this material. MC cutbacks with seventeen is second in preference. Tars, SC oils, Emulsions and AC of 30-100 penetration follow in preference.

The use of Emulsions is indicated to predominate in the field of Soil Stabilization, with eleven departments indicating its use. Tars with seven and MC cutbacks with five departments are also used.

The use of the materials which have been indicated does not disagree greatly with previous surveys on the use of these materials. Of particular interest is the expressed preference of RC material for surface treatments, and the fairly uniform preference expressed by the two intermediate and surface treatment types of construction for MC cutback.

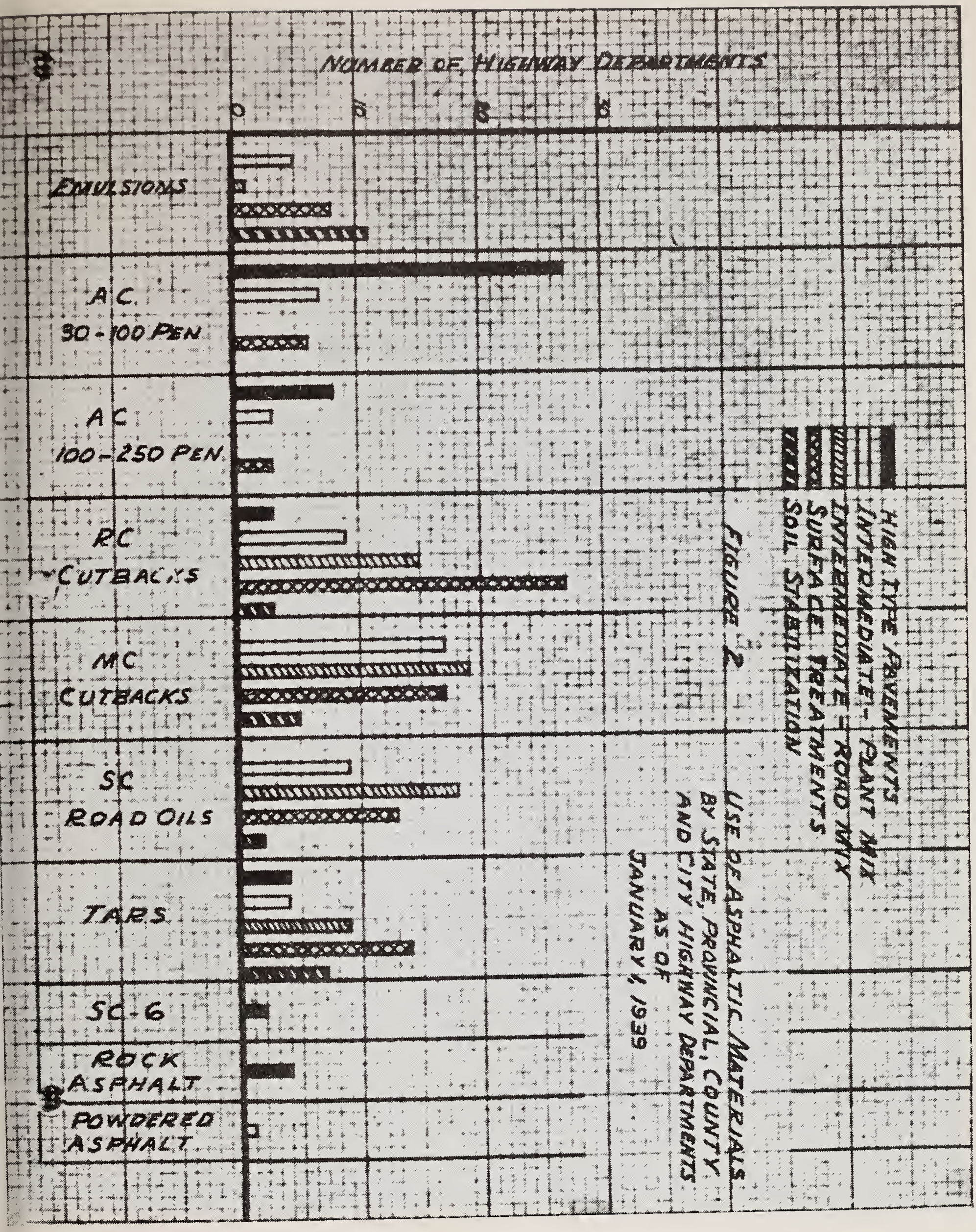


FIGURE 2

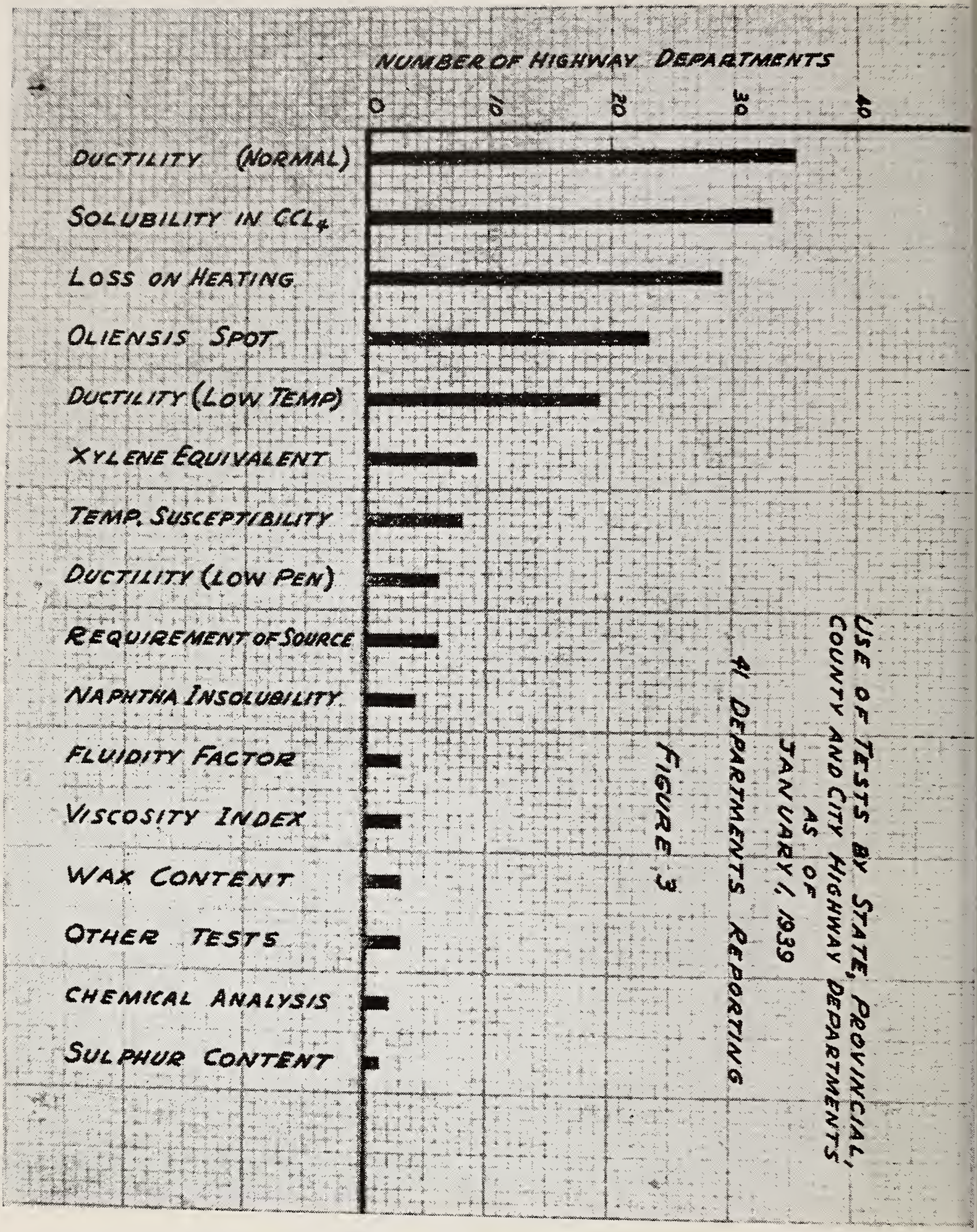
USE OF ASPHALTIC MATERIALS BY STATE, PROVINCIAL, COUNTY AND CITY HIGHWAY DEPARTMENTS AS OF JANUARY 1, 1939

Use of Tests

An analysis of the use of predicting tests, as indicated, is presented in graphical form in Figure 3.

Of import is the general (46% to 85%) use of one group of five tests, namely: Normal Ductility, Solubility in CCl_4, Loss on Heating, the Oliensis Spot Test, and Low Temperature Ductility. These tests appear to be the most used or preferred of the group of tests which have been considered predicting tests.

The Oliensis test, perhaps the most contriversial test in use, is recorded as used by 23, or 56% of the 41 departments reporting. A discussion of the test will be made in the following section.



USE OF TESTS BY STATE, PROVINCIAL, COUNTY AND CITY HIGHWAY DEPARTMENTS AS OF JANUARY 1, 1939

41 DEPARTMENTS REPORTING

FIGURE 3

Xylene Equivalent, Temperature Susceptibility, Low Penetration Ductility and Definite Requirement of Source are each recorded as preferred by more than 5 but less than 10 departments.

The remaining tests, Naphtha Insolubility, Fluidity Factor, Viscosity Index, Wax Content, Chemical Analysis, Sulphur Content and other tests, are each reported as preferred by less than 5 of the reporting group.

ANALYSIS OF TRENDS

Trends have been analyzed and presented in graphical form in Figure 4. In order to facilitate interpretation, and due to differences in type of construction which might indicate erroneous trends, the trends expressed by cities have been noted separately in the graph.

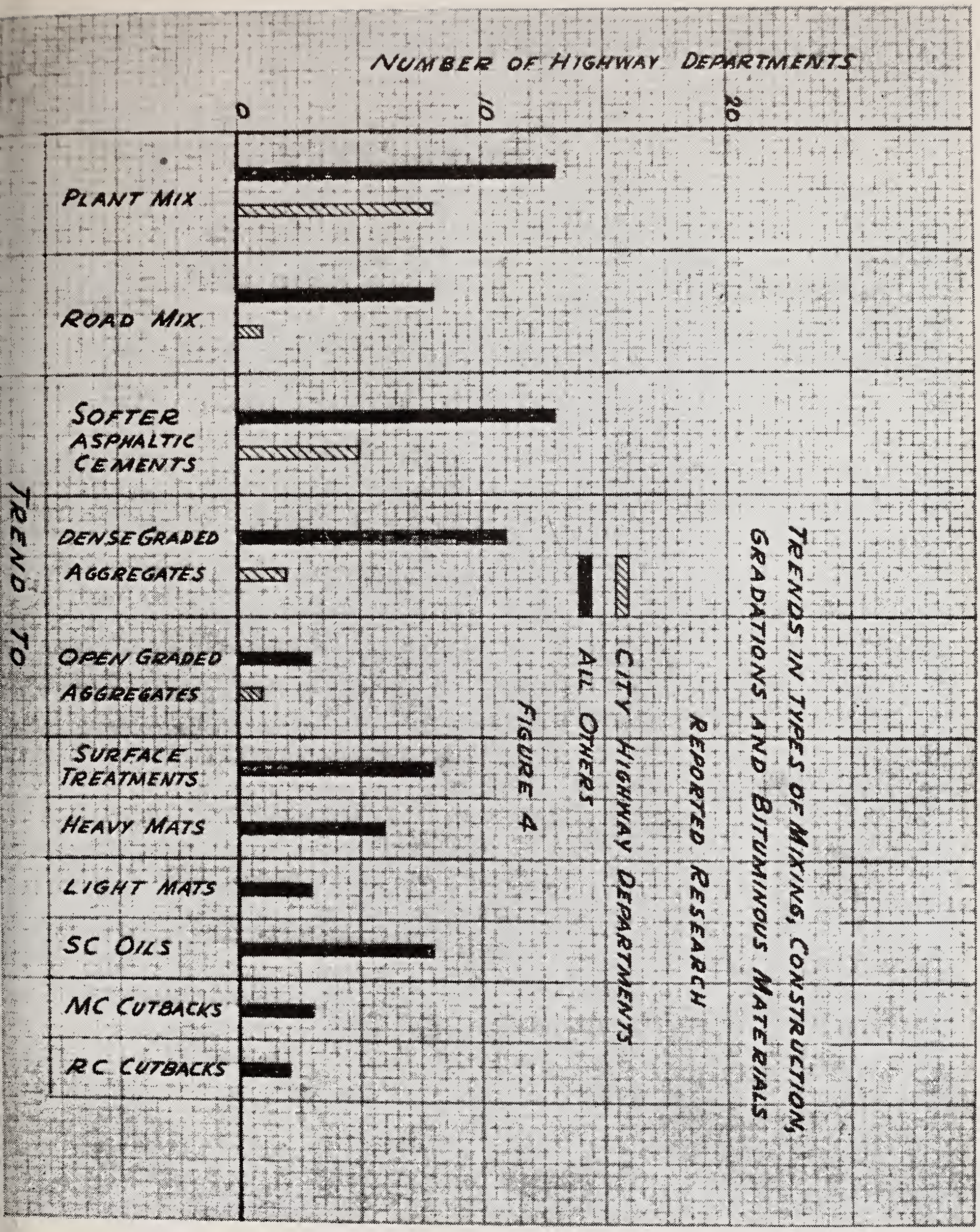


FIGURE 4

TRENDS IN TYPES OF MIXING, CONSTRUCTION, GRADATIONS AND BITUMINOUS MATERIALS

REPORTED RESEARCH

TRENDS IN MIXING

City trends are expressed as predominately plant mix with one city noting a road mix trend (for secondary roads). The expression of the use of plant mix as a trend may be somewhat in error, due to the wide use of this type of mixing in city pavement construction.

A new factor which is entering into city bituminous construction and which has widened considerably the types of construction here employed, is airport construction and maintenance. This subject has received considerable attention and has been well reviewed by Macatee[67]. Interest and diversification due to this type of construction will probably tend to increase.

A strong trend to plant mix is noted among all Highway departments other than cities. The many advantages of plant mix, together with the use of heavier asphaltic materials have increased this trend.

Trends in Aggregate Grading

A strong trend to dense graded mixes is indicated by all Highway departments other than cities. The latter group has long used the dense graded mixes. A trend noted by one city (Topeka) is the construction of open graded "black base."

The trends in aggregate gradation may be correlated to a certain extent to the conclusion reached in research on the durability of bituminous materials, as to the effect of dense graded mixes.

Trends to open graded mixes were indicated by three departments, California (special cases), Minnesota and McLennan County, Texas.

Mats and Surface Treatments

A diversification of trends is noted in surface types, with eight departments tending to Surface Treatments, six to heavy mats and three to light mats. These trends are probably influenced by a number of factors. The trend to Surface Treatments has been influenced to a considerable extent by base construction, which frequently is of the character where a light prime and armor coat provides an adequate surface.

Heavy mats may, to a certain extent, reduce subgrade deficiencies.

Light mats, noted by Montana as 1½ inch to 2 inch road mix dense graded mixes, and by Minnesota as ¾ inch mats of the same type, indicate in some areas a trend from Surface Treatments to a more substantial surface. North Carolina utilizes a tar prime, AC 150-200 penetration and one inch stone, followed by RC cutback and ⅜ inch stone seal.

Trends in Bituminous Materials

Two trends predominate the changes indicated by replies to this question, the trend to softer asphalt cements and to SC oils. The reason for these trends have been indicated in the previous discussion.

MC and RC cutbacks, with three and two departments, respectively, indicating preference, indicates that in this case the present use of these materials is broad and therefore not expressable as a trend.

ANALYSIS OF REPLIES ON RESEARCH

Three cities and 14 of the remaining groups indicated research on the durability of bituminous materials, varying from collection of observational data to involved technical investigations. Highway departments reporting research are as follows:

1. Arkansas—Field Observations.
2. California—Extensive laboratory research.
3. Iowa—Oxidation of bituminous materials.
4. Kansas—Extensive laboratory research.

5. Kentucky—Extractions and film adhesion.
6. Minnesota—Study of Asphalt Cements.
7. Montana—Extensive laboratory research.
8. Oregon—Effect of weathering and mixing on asphalt durability.
9. Rhode Island—Stripping tests.
10. South Dakota—Asphalt-aggregate mixes.
11. Utah—Experimental investigation using Gilsonite.
12. Washington—Stripping tests.
13. District of Columbia—Extensive laboratory and field research.
14. Detroit, Michigan—Effect of plant mixing conditions on asphalt.
15. Borough of Manhattan, N. Y.—Extractions and low temperature ductility.
16. Richmond, Virginia—Extractions and low temperature ductility.
17. McLennan County, Texas—Field Observations.

An analysis of the reported research indicates that investigations may in general be classified in six groups. These groups and the number of departments performing work in each group are as follows:

1. Extractions (and recovery)	8
2. Field observations	3
3. Stripping or adhesion	4
4. Oxidation or weathering	6
5. Low Temperature Ducility	2
6. Other phases	8

These results indicate that considerable attention is being paid to the behavior of asphalt in service as denoted by the number of departments employing extraction methods. Field observations are employed by three organizations.

The stripping of asphalt is still a current problem as is indicated by the number engaged in these investigations.

Oxidation or weathering of asphaltic material is being investigated by six departments.

Low Temperature Ductility research, noted by two cities, the Borough of Manhattan and the City of Richmond, indicates attention to one phase of the problem of pavement cracking, the behavior of the asphaltic material at low temperatures.

Other tests, which include work on base stocks, films and the effect of high temperatures in mixing operations, are being investigated by eight organizations.

The replies indicate considerable interest in the problems not only of the durability, but the use of asphaltic materials.

GENERAL SUMMARY OF REPLIES TO QUESTIONNAIRE

The general types of construction employed by the various organizations do not vary greatly, by type, although a great difference exists in the miles of each type constructed. The types of bituminous material used, with the exception of the large use of tars in eastern states and their lesser use in mid-western states, is predominately asphaltic materials. The general use of these materials does not vary greatly, although exceptional cases are occasionally noted.

The fact that five of the "predicting" tests are used by more than 46 per cent of the reporting groups, and that the Oliensis test is variously used by 56 per cent of the total group, is of considerable interest with respect to the use of these tests.

The trends toward the use of Plant Mixes, Dense Graded Aggregates, softer Asphalt Cements,, including SC materials (of high viscosity) and the trend to Surface Treatments are indicated.

The various trends are due in part to conclusions reached by research and experience, and probably in part to improved materials and methods of use.

Research on the durability of asphaltic material was noted by 17, or 42 per cent of the reporting groups, with other research noted but not recorded, pertaining to various other phases of asphalt use, such as Soil Stabilization. The proportion of departments investigating the durability of asphaltic materials indicates an extensive interest in this subject.

THE OLIENSIS TEST

The Oliensis Spot Test was not, according to Oliensis, intended for use as a quality test for asphalt. The use of the test for this purpose, since the test was described by Oliensis in 1933, has been accepted and found of value to some organizations, while others have not found the test to be entirely satisfactory for the purpose for which they were using it. That considerable difference in opinion exists is evident in an analysis of the comments received in replies to the questionnaire, noted briefly as follows:

Arkansas: No experience with materials which do not comply with Oliensis Spot test.

California: Adopted modified Oliensis Xylene equivalent procedure using normal heptane to guard against undesirable qualities in paving asphalt. (Not used on other asphalts).

Idaho: Not strictly a quality test and not justified in this area at the present time.

Iowa: Is required and should be required (on RC and MC) until something better is developed.

Louisiana: Will specify (as to whether positive or negative).

Montana: Use test as an indicator only, do not specify nor use as a basis for acceptance or rejection.

Nebraska: Oliensis test eliminated some unsatisfactory oils, also eliminated some satisfactory oils. Noted that positive oils with Xylene equivalent above 20 tended to increasing difficulty. One oil (Gulf Coast) with 60-80 Xylene equivalent was satisfactory.

Nevada: Have found that some asphalts being used were positive. Some of the jobs using positive material have given the best road mix and plant mix surfaces. Not a single failure traceable to oils or asphalts showing a positive test. Do not specify because test (at present, here) is not of sufficient importance.

N. Carolina: Have not adopted as we feel test would not be of any practical benefit to us on the bituminous materials which we use. (AC-RC-Tars).

S. Dakota: Have to depend on Oliensis Spot Test for a quality test. Seems now that greater amount of SC material needed to accomplish same results than formerly. Turns brown quicker.

City of Chicago: Consider the Oliensis Spot Test as a valuable clause in our specifications.

City-County of Denver: While Oliensis Spot Test holds out some good oils, the protection it gives us from inferior oils warrants keeping it in our specifications.

City of Detroit: We believe the Oliensis Spot Test to be of value as a qualitative test—quite significant in specifications. Since incorporating this test in our specifications, we have eliminated some new brands of material which have shown evidence through service, of being of low quality.

City of Milwaukee: Oliensis Spot Test O. K. in extreme cases but subject to controversy in border line cases.

New Brunswick, Canada: While we feel that the introduction of the Oliensis Test into our specifications would be technically desirable, we doubt whether it is feasible, from a practical point of view. Producers, can, by blending—produce asphalts which, while they do not meet the Oliensis Spot Test, are perfectly satisfactory in service.

Kansas: Require test to be run on all materials to determine whether positive or negative. Specifications may be open to both, or negative may be required as determined by proposal. Reasons follow in discussion.

These special comments may be arranged as follows:

Use of Test	No. of Depts.
Negative only	3
Will specify (positive or negative)	1
Will accept positive under added test	2
Test not used, not justified	2
Require neg. but better test desired	1
Used as indicator only	1
Positive is satisfactory, do not use test	2
Some positive is satisfactory, but use test	4

These opinions could, it is believed, be taken as a fairly good cross section of opinion regarding the value of the test. It is important to note that twelve of the above sixteen either directly or indirectly note satisfactory service from positive materials, or do not believe that the test should be applied. On the other hand, eleven use the test to some degree or for some purpose.

EXPERIENCE WITH THE OLIENSIS TEST IN KANSAS

The State Highway Commission of Kansas began investigations on the use of the Oliensis test in 1933, but conclusions regarding the test as published in a report[63] of the work on oils used in 1932 and 1933, were insignificant since only one material tested was negative Oliensis.

Since the original work, the percentage of negative material used has been appreciable, with both positive and negative material being used. From research of these materials, considerable data has been accumulated regarding their service behavior.

The research may be divided into two classifications, (1) the work on surface treatments where the asphaltic material is generally highly exposed, and (2) the work on bituminous dense graded mats where the asphaltic material is usually in a highly protected state. These are considered in the view of research information, as fundamentally different uses of the material.

The data pertaining to surface treatments was secured by observational methods, due to the difficulty of making laboratory tests on specimens from the road surface, and the necessity of evaluating certain factors of traffic, subgrade and construction. Laboratory tests are being made on prepared plates, from which extraction-recovery tests will later be made, but this data is not available at this time. The data on surface treatments was obtained in May, 1939, on surfaces constructed the previous year.

The method of obtaining data is indicated in Table 2, in which the first step to analysis has been taken, a grouping of the materials into types and classes. With further grouping into good, average and poor condition of the asphaltic material, a graphical analysis of the condition of the asphaltic material with respect to the various types and grades of material is possible. This graph is presented in figure 5.

By a further regrouping by classifying all material as to Oliensis spot and Kansas film specifications (Supplemental Specifications requiring clear film, 48 hours at 200° F., positive Oliensis only), the behavior of the asphaltic materials as to these tests may be analyzed. The results of the graphical analysis is shown in figure 6.

Figure 6 indicates that negative materials are fairly well balanced as to good, average and poor condition, with a tendency to good and average. (Ratio, good and average to poor equals 21/8.)

Positive material appears to be much more unbalanced than the negative material, with less good and average, and more poor conditions. (Ratio, good and average to poor equals 13/13).

TABLE 2

BITUMINOUS SURFACE TREATMENT SURVEY OF MAY, 1939

RECAPITULATION BY TYPES OF ASPHALT

No.	Type Asphalt	Spot	Route	County	Type Const.	Base	Agg. Struct.	Surface Grade	Surface Condition	Condition of Asphalt
1	AP3	N	US24	Clay	Seal	M	M	G		Soft—tenacious
14	AP3	N	K1	Pawnee	Pen.	N	M	G		Soft—tenacious
16	AP3	N	US183	Kiowa	Pen.	N	M	G	Bleed	Soft—tenacious
56	AP3	N	US160	Clark	Pen.	N	M	G	Bleed	Soft—tenacious
59	AP3	P	US59	Jefferson	Seal	BM	K	G		Soft—tenacious
13	AP3	P	K1	Rush	Pen.	N	M	G		Soft—tenacious
43	AP2	P	K18	Russell	Pen.	N	M	S		Good
47	AP3	P	K21	Graham	Pen.	N	M	G		Soft—tenacious
55	AP3	PF	US160	Meade	Pen.	N	M	G		Soft—tenacious
17	AP3	PF	US183	Edwards	Pen.	N	M	G	Bleed	Soft—tenacious
58	AP3	PF	US283	Clark	Pen.	N	M	G	Bleed	Very Soft—low ten.
9	RC4	N	K8	Russell	Pen.	N	M	U	Bleed	Soft
15	RC4	N	US283	Ford	Pen.	N	M	G		Soft—tenacious
18	RC4	N	FAS5A	Butler	Pen.	N	MK	U	Ravel	Dead
40	RC4	N	US83	Logan	Pen.	N	M	G		Soft—tenacious
42	RC4	N	US83	Norton	Pen.	N	M	G		Good
48	RC4	N	US83	Phillips	Pen.	N	M	G		Soft
49	RC4	N	US183	Phillips	Pen.	N	M	G		Soft
54	RC4	N	K23	Finney	Pen.	N	M	U	Ravel	Somewhat dead
5	RC4	P	K14	Mitchell	Pen.	N	M	G		Soft—tenacious
7	RC4	P	K14	Lincoln	Pen.	N	K	G	Bleed	Soft—tenacious
8	RC4	P	K18	Lincoln	Pen.	N	M	U	Bleed	Soft
11	RC4	P	K18	Russell	Pen.	N	M	U	Bleed	Soft
12	RC4	P	K4	Barton	Pen.	N	M	U	Bleed	Soft
44	RC4	P	US183	Rooks	Pen.	N	M	G		Good
46	RC4	P	US36	Cheyenne	Pen.	N	M	G		Soft
50	RC4	P	US183	Rooks	Pen.	N	M	S		Matrix hard
51	RC4	P	US36	Cheyenne	Pen.	N	M	S	Ravel	Matrix hard
53	RC4	P	US183	Ellis	Pen.	SB	M	S		Soft
57	RC4	P	K23	Lane	Pen.	N	M	U	Ravel	Dead
38	RC4	PF	US73	Brown	Pen.	N	M	U		Soft

TABLE 2

BITUMINOUS SURFACE TREATMENT SURVEY OF MAY, 1939
RECAPITULATION BY TYPES OF ASPHALT

No.	Type Asphalt	Spot	Route	County	Type Const.	Base	Agg. Struct.	Surface Grade	Surface Condition	Condition of Asphalt
30	RC3	N	K7	Crawford	Pen.	R	M	G		Soft—no tenacity
32	RC3	N	US160	Crawford	Pen.	R	M	U	Ravel	Soft—dead
33	RC3	N	US54	Woodson	Pen.	R	M	U	Ravel	Hard—dead
34	RC3	N	US75	Woodson	Seal	M	K	G		Ten. ductile, strong
35	RC3	N	US75	Woodson	Pen.	R	M	G		Soft—tenacious
36	RC3	N	US75	Osage	Pen.	R	M	G		Soft—not tenacious
39	RC3	N	US73	Brown	Pen.	N	M	U	Bleed	Soft
10	RC3	N	K4	Jefferson	Pen.	R	M	G		Soft, fairly ten.
28	RC3	P	US59	Labette	Pen.	SB	MK	G		Soft to dead & greasy
29	RC3	P	US160	Labette	Pen.	SB	M	G		Soft to greasy, not tenacious
31	RC3	P	US160	Crawford	Pen.	SB	MK	G		Soft—no tenacity
52	RC3	P	US36	Rawlins	Pen.	N	M	S		Hard
22	RC2	N	US75	Montgomery	BDT	BM	K	G		Ten.—fairly soft
19	MC4	N	K11	Lyon	Pen.	R	M	U	Bleed	Soft, poor adhesion
21	MC4	N	US160	Montgomery	Pen.	R	K	U	Ravel	Soft to dead
26	MC4	N	K96	Butler	Pen.	R	M	U	Bleed	Soft
27	MC4	N	US77	Butler	Pen.	R	M	U	Bleed	Soft to hard & dead
2	MC4	P	US81	Republic	Pen.	R	M	G	Bleed	Soft to hard—greasy
3	MC4	P	US36	Republic	Pen.	R	M	S	Bleed	Soft to dead
61	MC4	P	US36	Republic	Pen.	N	M	U	Ravel	Dead
20	MC3	N	US77	Butler	Pen.	R	M	U	Bleed	Soft
41	MC3	N	US40	Logan	Seal	M	M	G		Good
60	MC3	N	K39	Wilson	Pen.	R	M	U	Ravel	Soft to dead
23	MC3	P	US166	Chautauqua	Pen.	R	M	U		Brittle—dead
25	MC3	P	US166	Cowley	Pen.	R	M	U		Brittle—dead
45	MC3	P	US40	Gove	Seal	M	M	U	Ravel	Hard, dry, brittle
4	MC3	PF	US36	Jewell	Seal	M	M	S	Bleed	Soft
24	MC3	PF	US75	Wilson	Pen.	R	M	G		Soft—tenacious
37	MC2	PF	US73	Atchison	Pen.	N	M	G		Soft

Key to Observations

Spot:

- N—Negative Oliensis.
- P—Positive Oliensis.
- PF—Positive Oliensis meeting Kansas film test.

Type of construction:

- Seal—Seal.
- Pen.—Oil penetration and Armor Coat.
- BDT—Bituminous Drag Treatment.

Base:

- M—Bituminous mat.
- N—Previously unoiled.
- BM—Bituminous Macadam or similar construction.
- R—Reconstructed surface.

Aggregate Structure:

- M—Matrix, asphalt plus fine material plus aggregate.
- K—Keyed, fairly coarse aggregate with free asphalt.
- MK—Partially matrix, partially keyed.

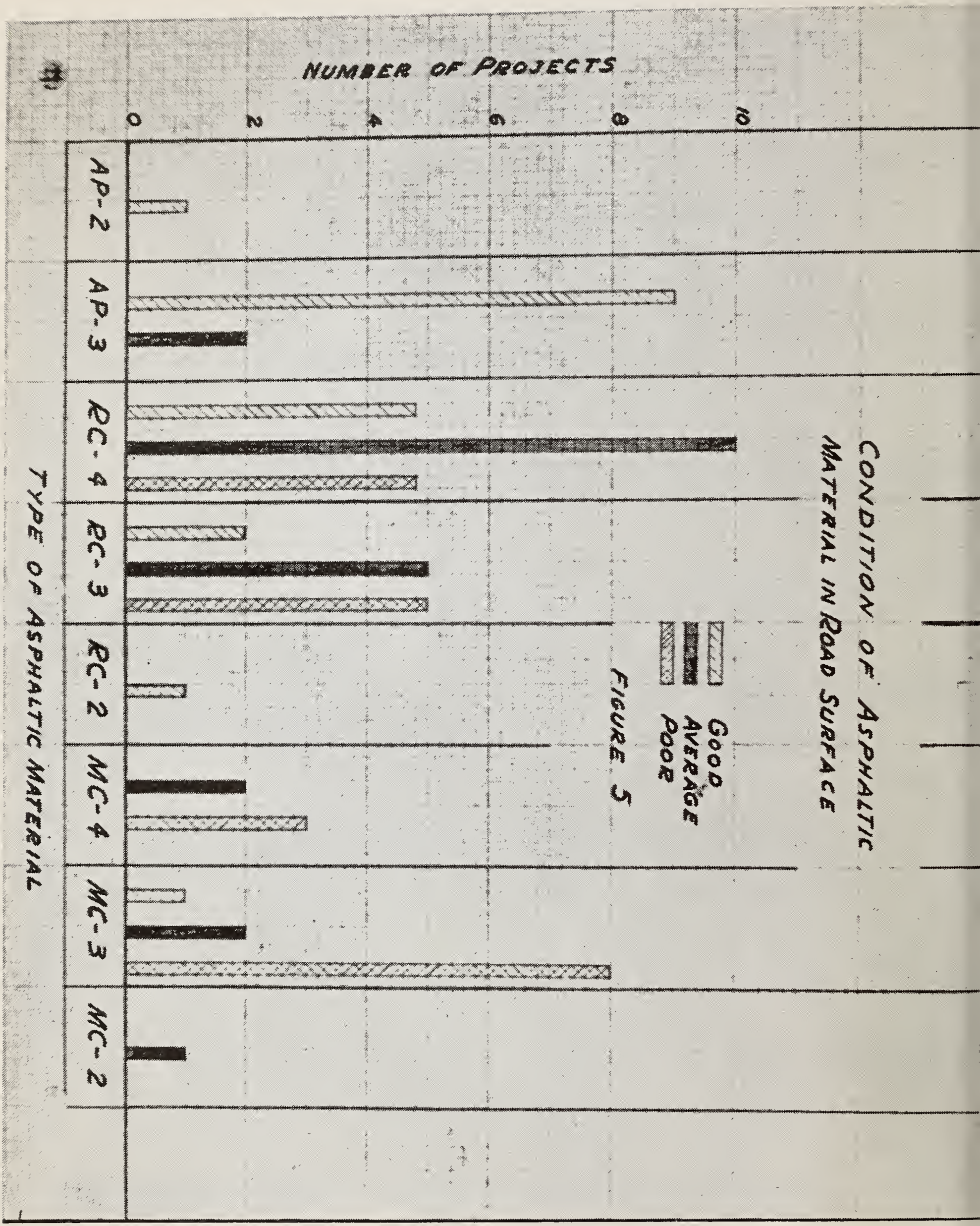
Surface Grade:

- G—Good, does not have permanent injury, or the unsatisfactory condition such as bleeding, has been satisfactorily corrected.
- S—Satisfactory, will probably give satisfactory service but is not as good as it should be.
- U—Unsatisfactory, denotes permanent injury or as yet unsatisfactory correction of distress at time of inspection.

Incidental to the Oliensis test is data on the behavior of positive film (PF) materials, which are indicated as possibly presenting behavior superior to the general positive materials. This is an indication only, however, and does not represent here sufficient data for definite conclusions on this type of material.

For the purpose of information relative to other parts of this paper, a third analysis, figure 7, based on the average condition of the surface of the surface treatment projects with respect to three general types of asphaltic material has been included. Since this does not directly include asphalt behavior, a check is secured on the analysis in figure 5.

A summary of the comparative behavior and negative Oliensis asphalts from their observed behavior in highly exposed surface treatments indicates that negative asphaltic materials, insofar as their consistency, adhesiveness and cohesiveness may be determined by observation, are superior to the positive materials.



CONDITION OF ASPHALTIC MATERIAL IN ROAD SURFACE

FIGURE 5

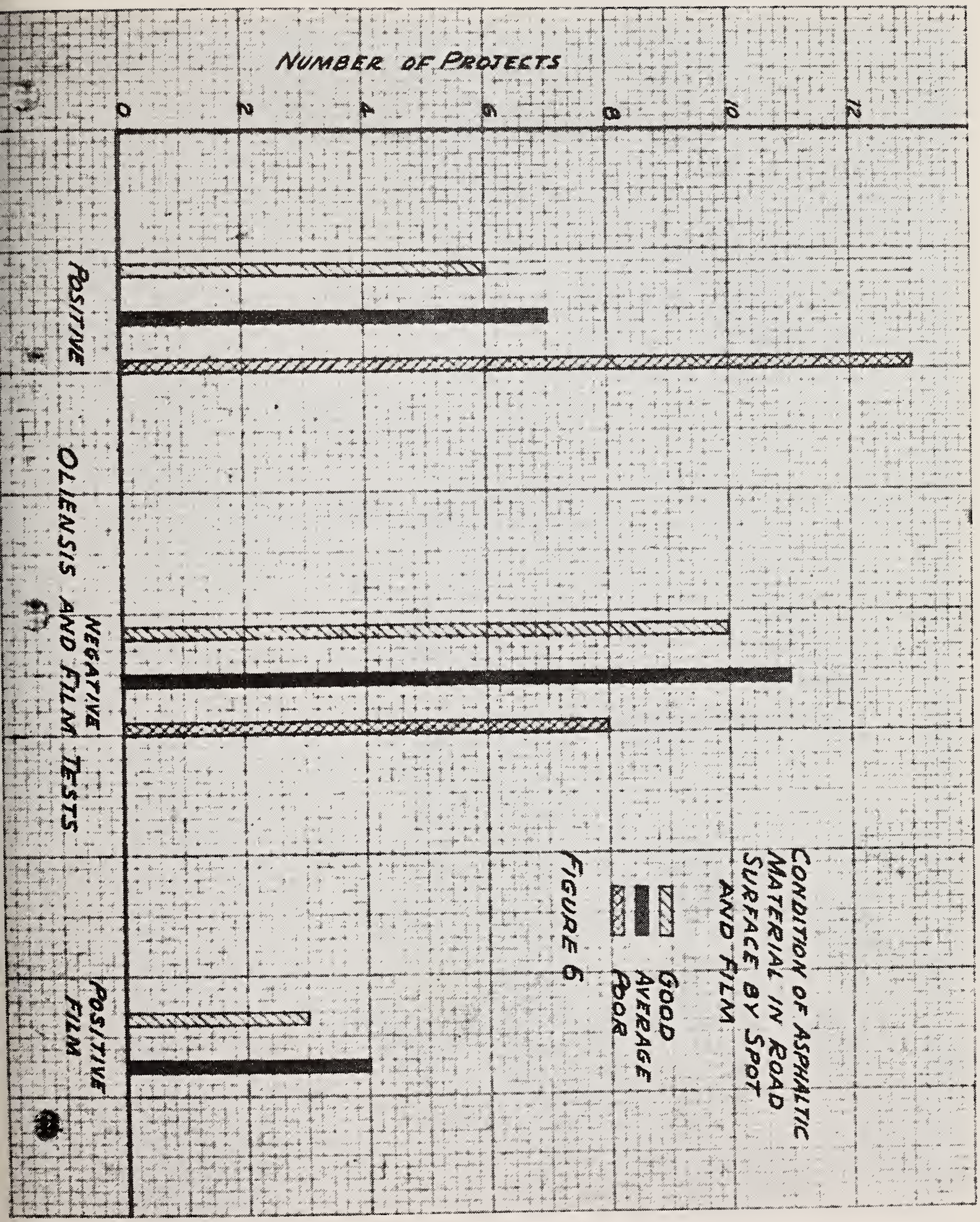
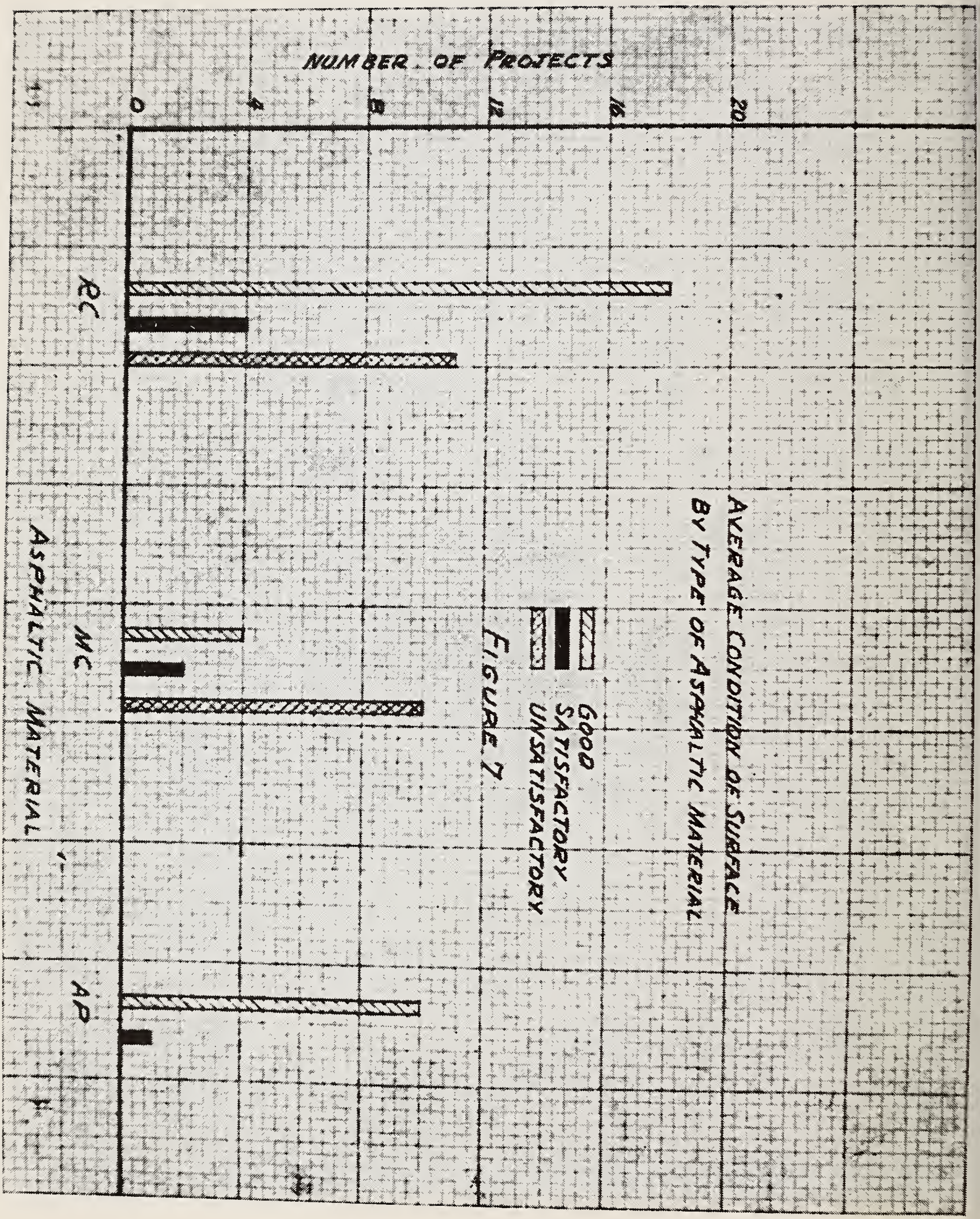

NUMBER OF PROJECTS
0
2
4
6
8
10
12
POSITIVE
NEGATIVE
POSITIVE FILM
OLIENSIS AND FILM TESTS
CONDITION OF ASPHALTIC MATERIAL IN ROAD SURFACE BY SPOT AND FILM
GOOD
AVERAGE
POOR

FIGURE 6



AVERAGE CONDITION OF SURFACE BY TYPE OF ASPHALTIC MATERIAL

FIGURE 7

The second classification of research, the work on dense graded mats in service, using extraction-recovery methods, also throws some light on the behavior of the respective kinds of materials. Extraction data from fourteen bituminous mat projects selected from a number of such data on the basis of greatest period of time between construction and last extraction test, and on one type of material, MC cutback, have been tabulated in Table 3. Seven of these projects are negative and seven positive Oliensis materials.

All of the materials, whether positive or negative, indicate comparatively large drops in both penetration and ductility, and rises in softening point with the lapse of time. By comparing the total drops in penetration and ductility, (using 110 cm. as a base for the latter) with the elapsed time between the original test and last extraction, interesting information is secured. The results may be tabulated as follows:

	Time in Mos.	Total Pen. Drop	P.D. per Mo.	Total Duct. Drop	Duct. Drop Per Mo.
Positive	181	681	3.76	582	3.2
Negative	147	539	3.66	468	3.19

TABLE 3

EXTRACTION AND RECOVERY TESTS

Positive Oliensis Materials

County	Constructed Samples	Type Asphalt	Age Mo.	Pen. 77°	Duct. 77°	Soft Point (B. & R.)	Spot
5	1936	MC4	0	126	110+	......	Pos.
	1937		9	40	110+	123	Pos.
	1938		27	29	110+	130	Pos.
29	1937	MC4	0	135	110+	......	Pos.
	1937		3	60	61	126	Pos.
	1938		14	53	29	133	Pos.
29	1937	MC4	0	135	110+	......	Pos.
	1937		3	63	91	125	Pos.
	1938		14	35	7	151	Pos.
55	1936	MC3	0	111	110+	......	Pos.
	1937		10	33	14	142	Pos.
	1938		28	32	5	157	Pos.
73	1935	MC4	0	151	110+	......	Pos.
	1936		12	19	17	152	Pos.
	1937		21	15	4	163	Pos.
	1938		40	14	3	171	Pos.
80	1935	MC4	0	109	110+	......	Pos.
	1936		10	32	38	142	Pos.
	1937		20	33	21	146	Pos.
88	1935	MC4	0	128	100+	......	Pos.
	1936		11	37	100+	......	Pos.
	1937		20	36	100+	128	Pos.
	1938		38	18	13	140	Pos.

TABLE 3

(Continued)

EXTRACTION AND RECOVERY TESTS

Negative Oliensis Materials

County	Constructed Samples	Type Asphalt	Age Mo.	Pen. 77°	Duct. 77°	Soft Point (B. & R.)	Spot
14	1936	MC3	0	150	110+	......	Neg.
	1937		10	66	110+	129	Neg.
	1938		27	79	110+	128	Neg.
29	1937	MC4	0	133	110+	......	Neg.
	1937		3	96	110+	121	Pos.
	1938		14	64	82	129	Pos.
29	1937	MC4	0	129	110+	......	Neg.
	1937		3	72	110+	125	Neg.
	1938		14	51	37	151	Pos.
55	1936	MC3	0	132	110+	......	Neg.
	1937		9	60	92	128	Neg.
	1938		27	53	38	137	Neg.
80	1935	MC4	0	113	110+	......	Neg.
	1936		10	40	27	147	Pos.
	1937		20	26	8	158	Pos.
81	1936	MC3	0	129	110+	......	Neg.
	1936		2	132	102	118	Neg.
	1937		8	76	28	128	Neg.
	1938		20	70	28	129	Neg.
	1938		25	56	15	138	Neg.
80	1935	MC4	0	113	110+	......	Neg.
	1936		10	37	80	141	Pos.
	1937		20	31	12	150	Pos.

These results, when interpreted in terms of unit drop per month over a period of time do not indicate a decided superiority of one kind of material over the other. An examination of the minimum penetrations and ductilities does not indicate large differences if similar periods of time are considered. Extraction-recovery data therefore does not indicate great differences in behavior between positive and negative materials when the materials are enclosed in highly protective aggregate structures. (It is indicated that some other reaction is of much more importance to the material noted here, than the characteristics indicates by the Oliensis test).

Evidence supported by data which has been presented indicates that where the two kinds of material (positive and negative) are highly exposed over a period of time as in surface treatments, the negative materials have a degree of superiority over positive materials. Where the materials are highly protected in undisturbed aggregate structures, very little difference apparently exists between the aging behavior of positive and negative material of the type and during the periods of time noted.

FACTORS AFFECTING TEST

It has been noted in the work of a number of investigators including the work of the writer on translucent asphaltic films, that many of the positive Oliensis materials have tendencies to change or deteriorate rapidly when exposed to air, heat or various accelerating agencies. On the other hand, correlation of service behavior to the noted laboratory test deficiencies is frequently difficult to secure in that materials of poor laboratory behavior may give good service in road surfaces, while some materials with apparently superior laboratory test qualities, may give relatively poorer service. It is indicated that the reasons for the difficulty in securing laboratory test-serviceability correlations may be due to a number of factors which include the following:

a. The development in service of defects in the asphaltic material which were not indicated in the laboratory test (internal structure change, polymerization, high thixotropic change, etc.), these defects varying largely, apparently, with type of material as varied by fluxes or diluents.
b. The blanketing or retardation of the unfavorable reactions noted in the laboratory tests (rapid loss and hardening due to volatilization, oxidation, polyermization or cracking at high temperatures, structure changes due to heat and air, etc.) by the protective action of the aggregate structure, assisted by lower temperatures. This behavior varies according to use of the asphaltic material.
c. A difference in the service behavior of materials meeting a given test when the materials are derived from different sources (fields).
d. A difference in the service behavior of materials meeting a given test when the materials are produced by different manufacturing processes (maximum temperature, pressure, time, etc.)
This factor, manufacture, is frequently of considerable importance.
e. Erratic service behavior when materials approach the border line of the test conditions.

These test-service behavior factors apply in general to a large number of tests, but they also apply particularly to the Oliensis test. With these factors in mind, it is evident that very good reasons exist for the differences in opinion and valuation which have been placed on the Oliensis test.

CONCLUSIONS — THE OLIENSIS TEST

The comments which have been made by the representatives of the various groups and departments in the questionnaire survey and the research data from Kansas experience with the Oliensis test, indicate that a number of conclusions as to the present use and behavior of the Oliensis test may be made. Among these may be enumerated the following:

1. The Oliensis test divides asphaltic material into two kinds of material.
2. The two kinds of material usually have other distinguishing differences of character besides the Oliensis reaction.
3. These differences in character usually tend to become less as the materials approach a change from negative to positive, or vice versa.

4. Neither of the two kinds of material have all good or all bad characteristics.
5. Positive materials, in general, when exposed in thin films tend to be more susceptible to weathering than negative materials.
6. Some positive materials if properly protected, mainly from heat and air, may yield equal or better serviceability than a corresponding negative material.
7. The value of the Oliensis test as to the durability of the asphalt may be expected to vary according to type of material, use, source, and manufacture.
8. It is believed that the Oliensis test or variations based upon this test are inherently valuable if proper cognizance is taken of the limitations of the test and if the use of such tests is based on careful observation, experience and research as pertaining to the particular condition of use existing in the group or organization employing the test.
9. It is indicated that the class of materials designated as positive may have important applications of use if the kind is controlled by further tests and the material is used in a manner consistent with its properties.
10. It is indicated that the class of materials designated as negative may have important properties differing from positive materials which would affect appreciable economics in certain types of construction and perhaps make possible new type of construction.
11. Due to the many factors which influence the behavior of bituminous materials, it appears that at this time no specific rule regarding the Oliensis or similar test is possible, but that the use of such test or tests must depend to a large extent upon comparatively localized conditions and experience.

SUMMARY

The writer has endeavored in this paper to review the problem of durability of asphaltic materials and the relationship of that durability to the serviceability of pavements and road surfaces constructed with these materials.

The major trends of change and research seeking greater durability and the apparent advantages and disadvantages of the trends and methods have been briefly indicated.

The discussion of the Oliensis test has been based largely upon available research, the communications received in reply to the questionnaire survey, and experience with the test in Kansas during the past six years. Complete concurrence on the opinions expressed on the Oliensis test is not expected. It is believed that only by a difference of opinion and the statement of these differing opinions to others and by others, can the true value of the test be established. The use of the tests, as noted by the number of departments using it, has become an important economic factor in the production and use of asphalt.

ACKNOWLEDGEMENT

The writer wishes to express grateful appreciation to the many participants who so kindly gave their time to completing and returning the questionnaires submitted in the survey of construction and research. Particular appreciation is expressed to those who volunteered special comments on the Oliensis and similar tests.

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Figure 1

REACTIONS BY WHICH ASPHALT MAY DECREASE IN SERVICEABILITY

Accelerating Factors are Noted Under the Reaction

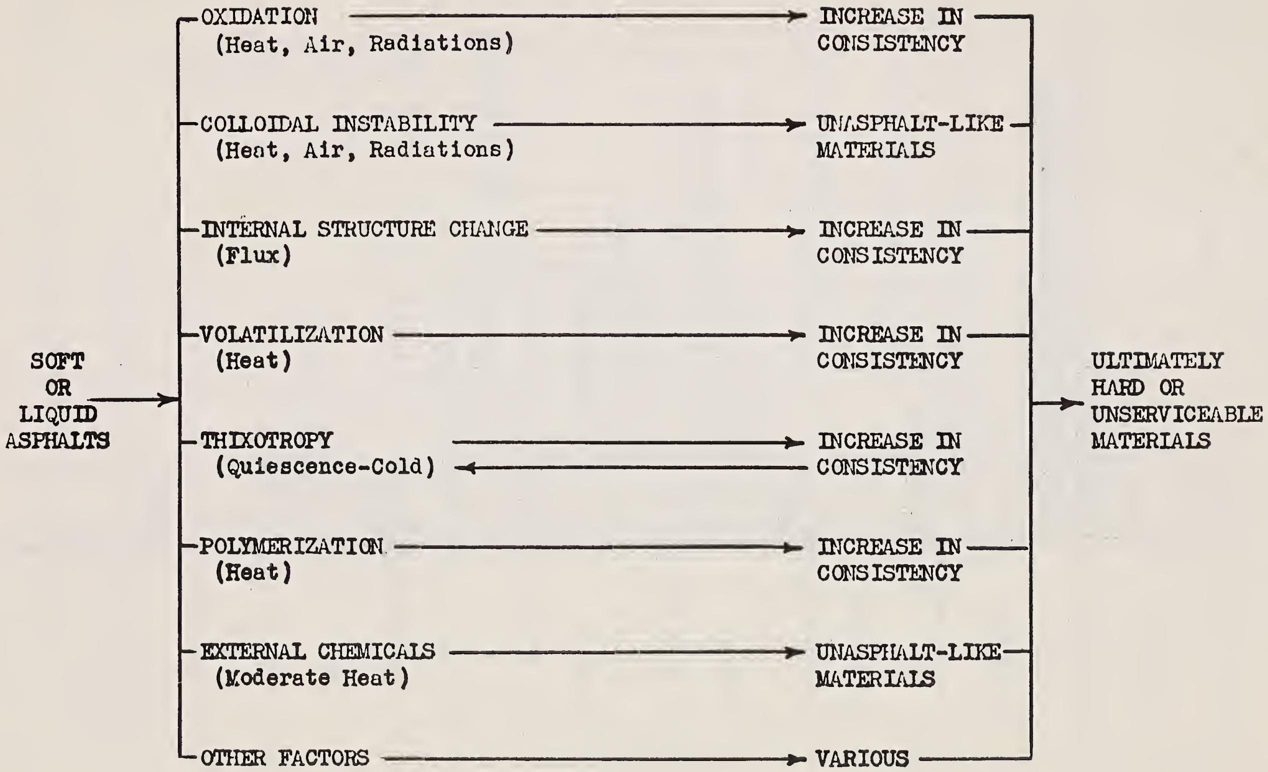
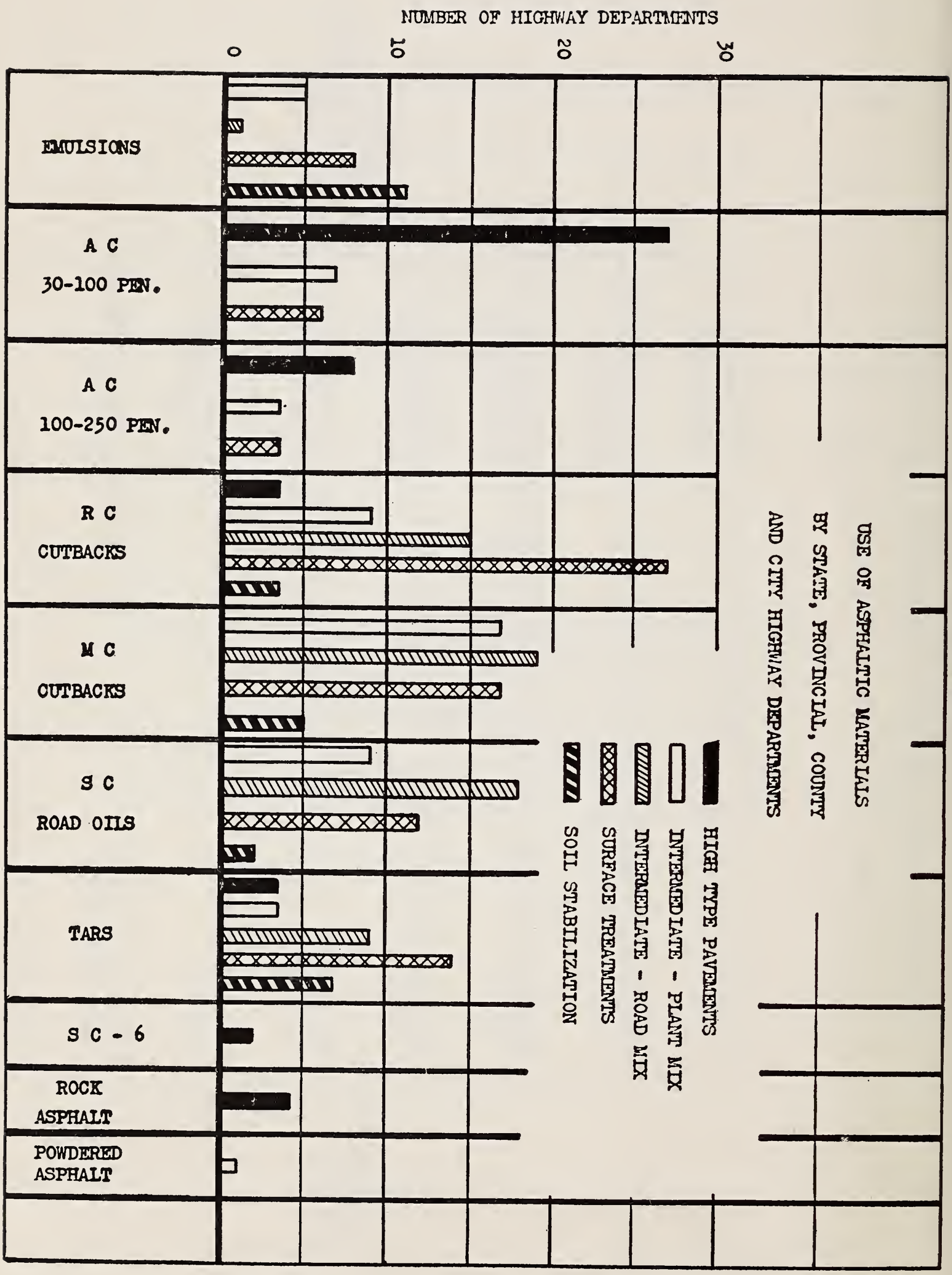
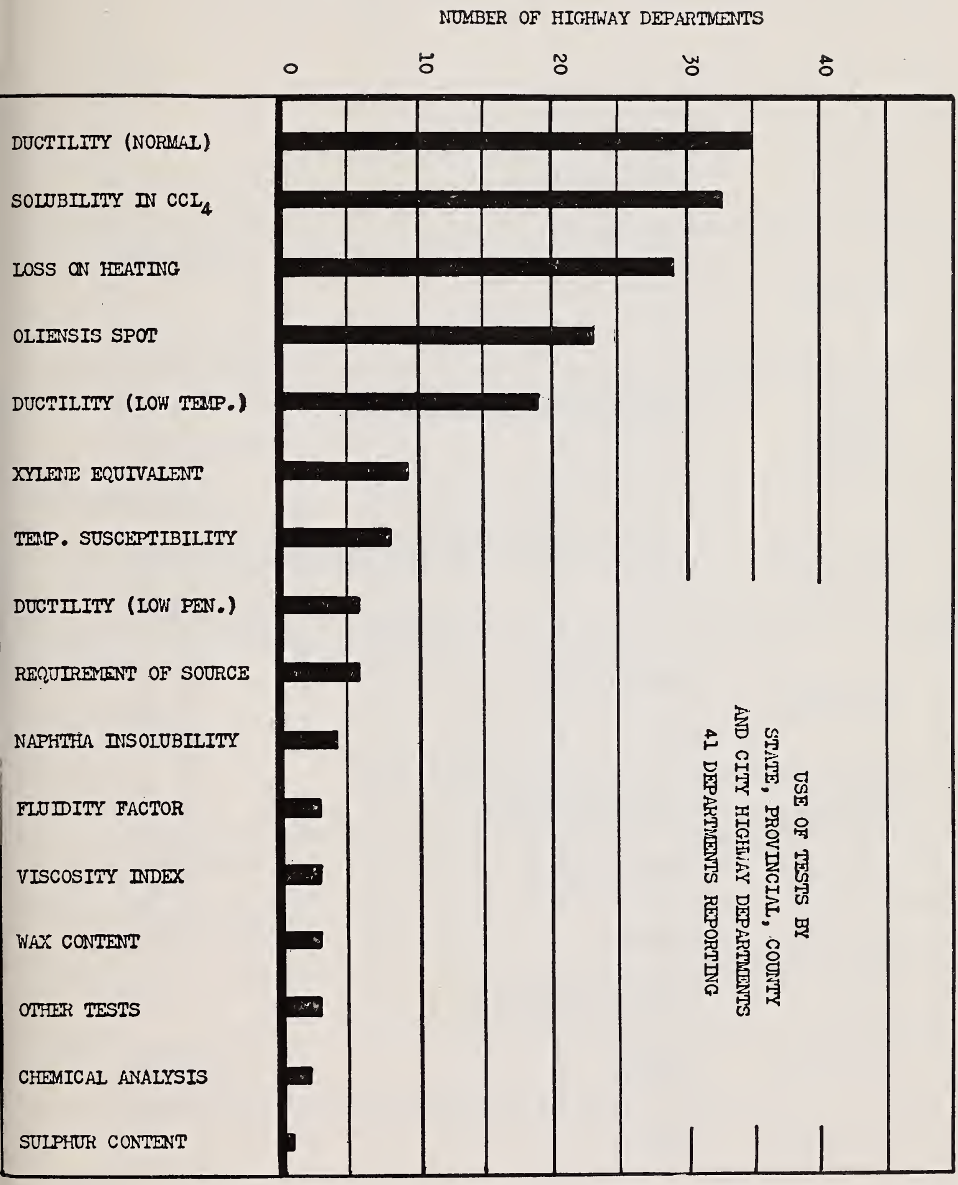

SOFT OR LIQUID ASPHALTS
OXIDATION (Heat, Air, Radiations)
INCREASE IN CONSISTENCY
COLLOIDAL INSTABILITY (Heat, Air, Radiations)
UNASPHALT-LIKE MATERIALS
INTERNAL STRUCTURE CHANGE (Flux)
INCREASE IN CONSISTENCY
VOLATILIZATION (Heat)
INCREASE IN CONSISTENCY
THIXOTROPY (Quiescence-Cold)
INCREASE IN CONSISTENCY
POLYMERIZATION (Heat)
INCREASE IN CONSISTENCY
EXTERNAL CHEMICALS (Moderate Heat)
UNASPHALT-LIKE MATERIALS
OTHER FACTORS
VARIOUS
ULTIMATELY HARD OR UNSERVICEABLE MATERIALS

FIGURE 2



USE OF ASPHALTIC MATERIALS BY STATE, PROVINCIAL, COUNTY AND CITY HIGHWAY DEPARTMENTS



USE OF TESTS BY
STATE, PROVINCIAL, COUNTY
AND CITY HIGHWAY DEPARTMENTS
41 DEPARTMENTS REPORTING

FIGURE 3

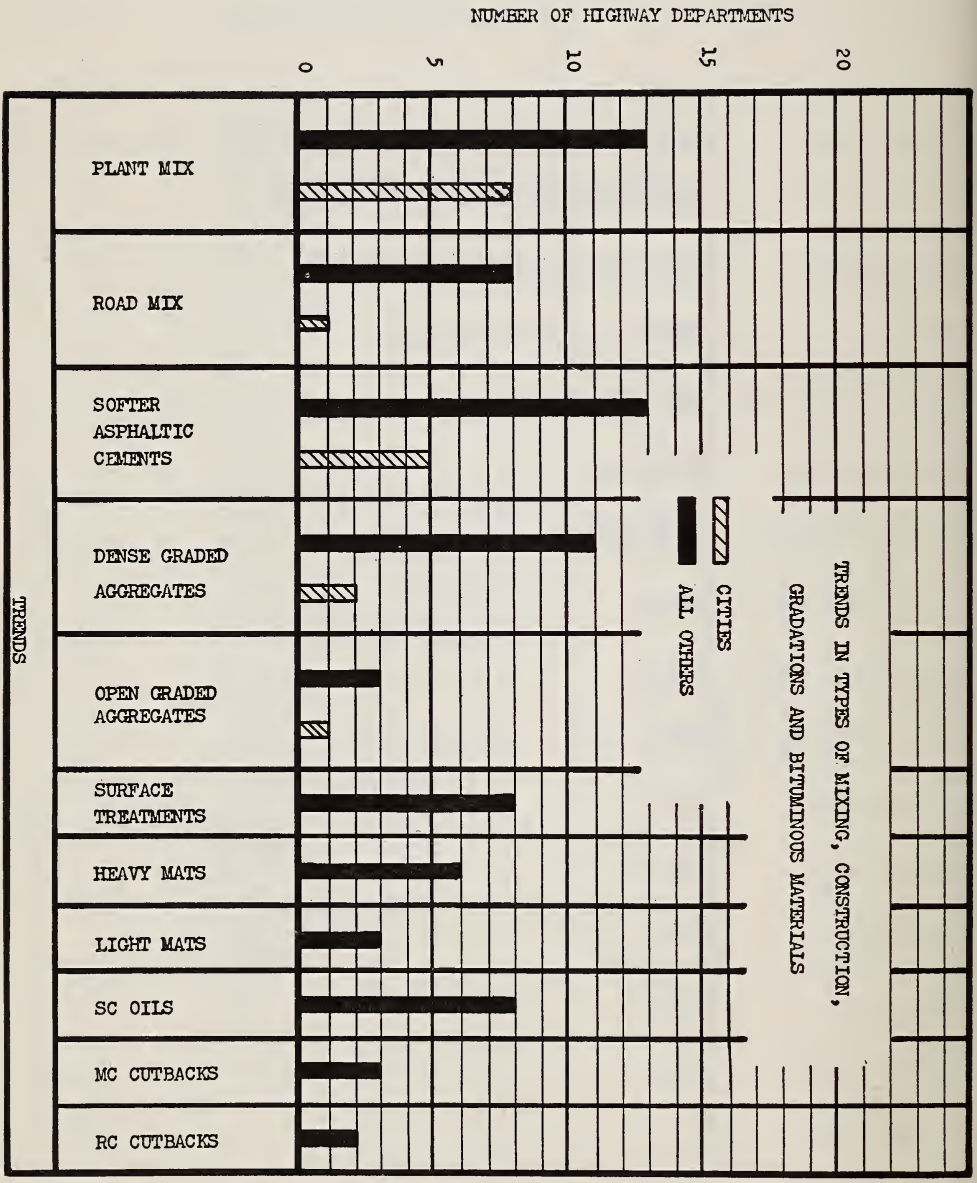
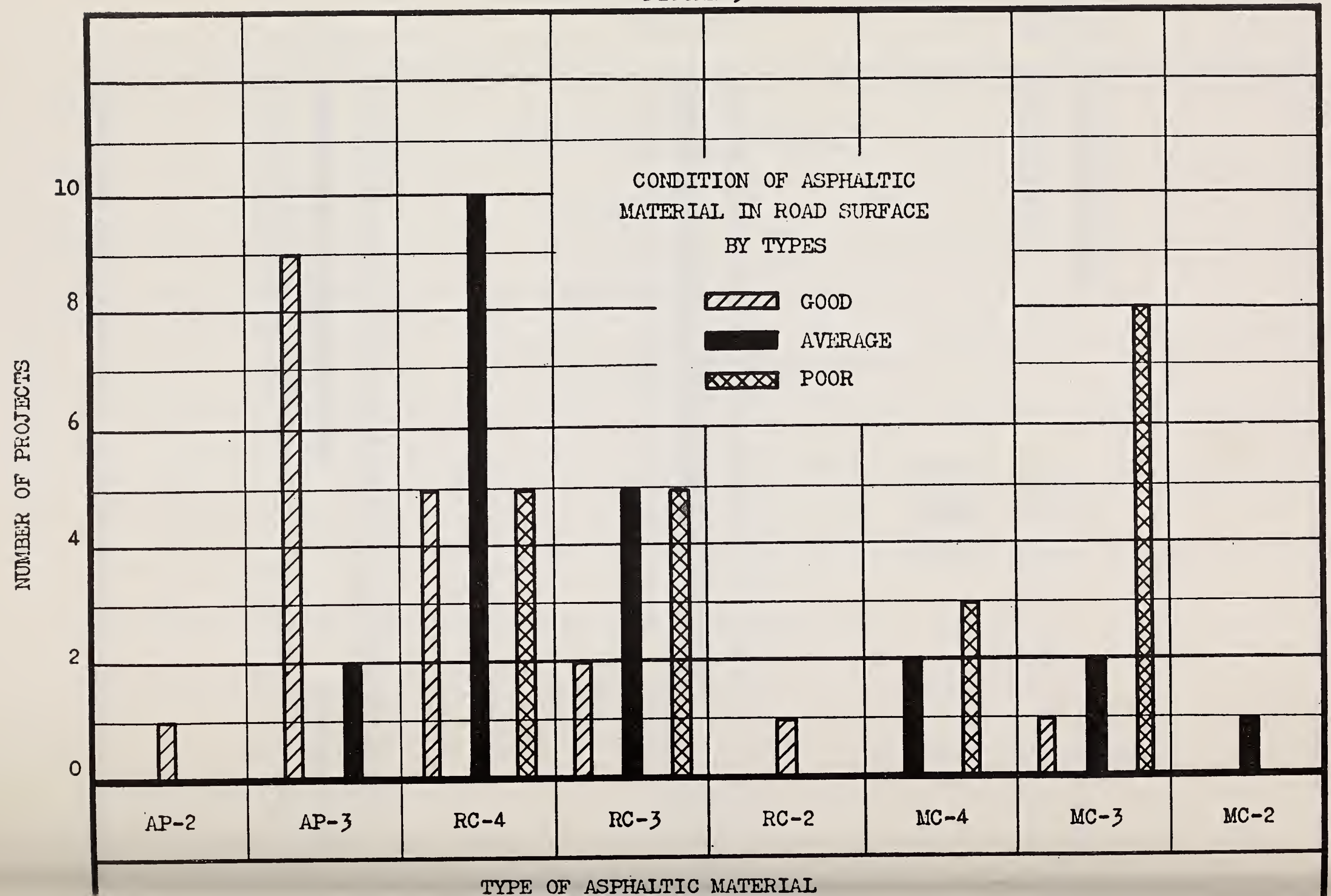


FIGURE 4

TRENDS IN TYPES OF MIXING, CONSTRUCTION, GRADATIONS AND BITUMINOUS MATERIALS

FIGURE 5



CONDITION OF ASPHALTIC MATERIAL IN ROAD SURFACE BY TYPES

FIGURE 6

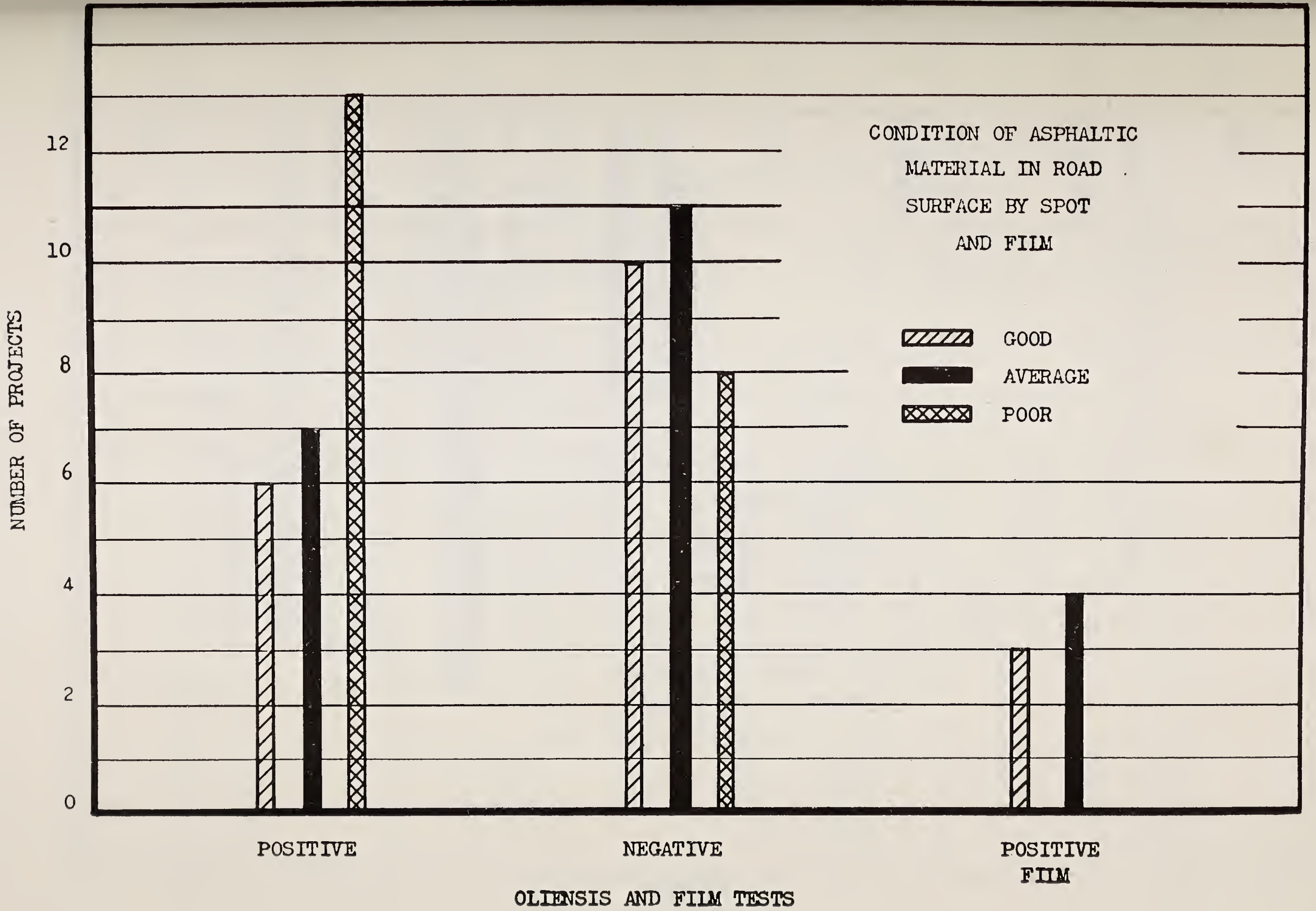
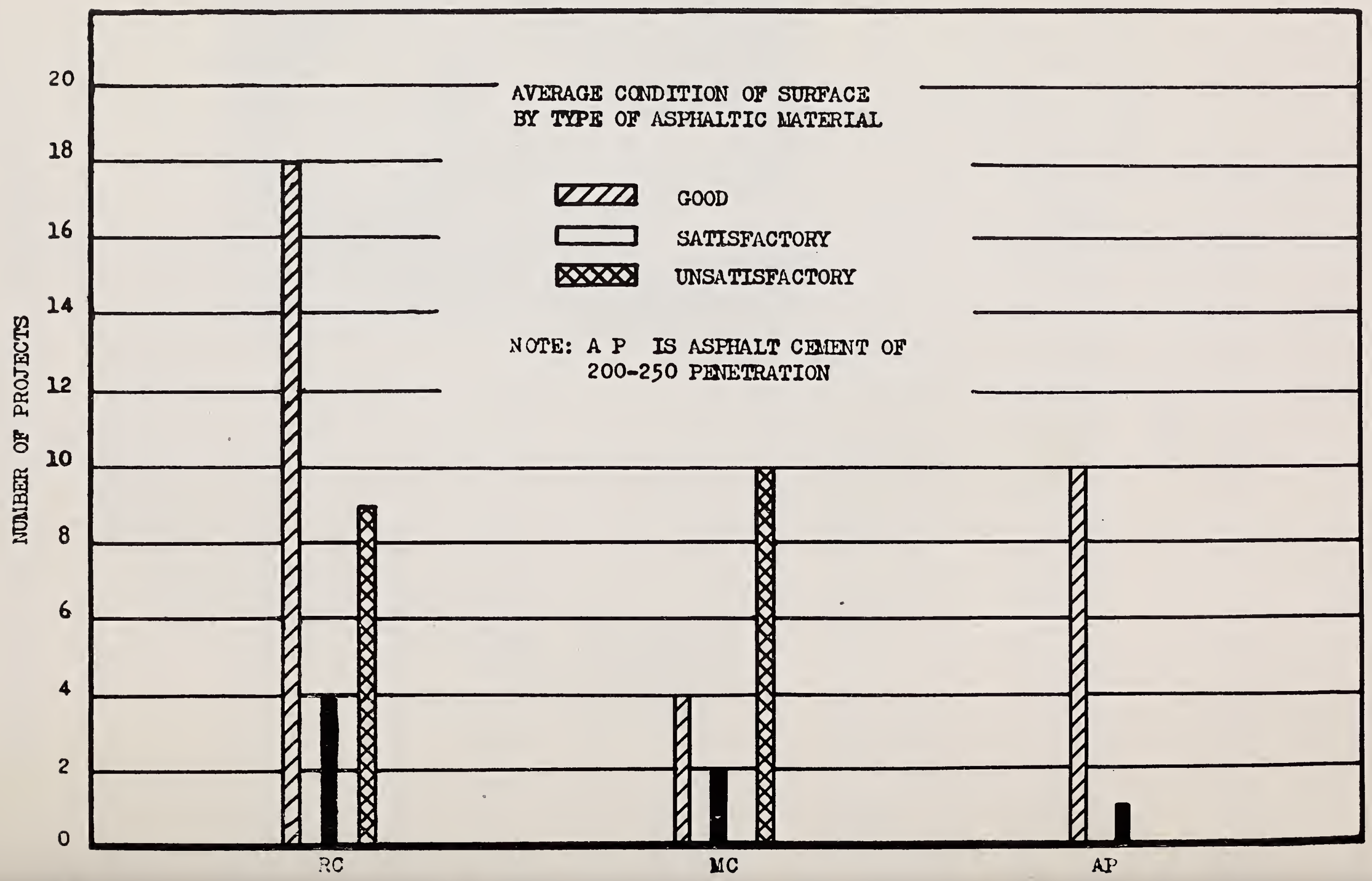

CONDITION OF ASPHALTIC
MATERIAL IN ROAD
SURFACE BY SPOT
AND FILM
GOOD
AVERAGE
POOR
NUMBER OF PROJECTS
12
10
8
6
4
2
0
POSITIVE
NEGATIVE
POSITIVE
FILM
OLIENSIS AND FILM TESTS

FIGURE 7


AVERAGE CONDITION OF SURFACE BY TYPE OF ASPHALTIC MATERIAL
GOOD
SATISFACTORY
UNSATISFACTORY
NOTE: A P IS ASPHALT CEMENT OF 200-250 PENETRATION
NUMBER OF PROJECTS
0
2
4
6
8
10
12
14
16
18
20
RC
MC
AP

Foundations for Bituminous Surfaces

By

O. L. STOKSTAD
Soils Engineer
Michigan Highway Department

The technique of designing and buil ing bituminous highways is now quite ge erally based on the principle that t finished road is only as good as its fou dation. In accordance with this concep the foundation must have sufficient streng to support the wheel loads to which t road will be subjected. The surface serv as a protection for the foundation again weather and the abrasive action of tra fic.

Failure to recognize this principal r sults in excessive maintenance costs, unle the surface is constructed sufficient strong to bridge foundation weaknesses. highway thus constructed can usually longer be classified as low cost.

The field experience on which th paper is based, has been limited to the glacial and lacustrine regions the Great Lakes. For the most part this is a region of wide variatio in soil. A careful study and classification of its soils is therefore produ tive of a wide range of information useful in developing a well balanc highway design.

The component parts of a bituminous highway may be classifi as (1) Site, (2) Subgrade, (3) Sub-base, (4) Base and (5) Surface. indicated by the title, this paper will be concerned with the first fo of these divisions.

SITE

A study of site involves the landscape environment with which hig way engineers must deal. In this study it is convenient to use son form of soil classification which reflects a large amount of general i formation. In connection with land use, for instance, should agricultur use be encouraged by the improvement or construction of farm to mark roads, or should highway funds be withheld in order that submargin land may be returned to other more profitable uses?

A soil survey of the site will indicate the extent of construction whi will be necessary and the type of surfacing, which is likely to be mo successful. In the past, many mistakes have been made in assuming, because a gravel road was carrying traffic with a fair degree of success, that a bituminous surface would necessarily result in an improvement.

The soil survey should contain information as to the amount of swamp area to be crossed. Sufficient sounding information should be included so that the type and expense of construction may be estimated. Failure to supply such soil information is responsible for most human interest stories published concerning the disappearance of a section of road into a so called under-ground lake. The history of these instances are all quite similar. Without checking foundation conditions fill materials are hauled in from the nearest high ground. Usually the failure does not start until the new embankment is practically finished. The road may disappear over night, leaving in its place on open area of water along each side of which are freshly heaved banks of peat. Then if the fill material consists of clay, as is often the case, the sink-hole may require twice the normal earth quantities to complete the embankment—an embankment which will continue to warp, shift and settle for years to come.

Surely the soils of the site should be classified so that general problems to be encountered may be anticipated before construction begins. The common excuse that the job is of such secondary nature that the cost of a soil survey is not warranted, is no longer valid. Techniques have been developed which are sufficiently simple and economical to be useful in connection with low cost highway construction. Finally the soil survey is a permanent record available at any time should service behavior studies become important.

SUBGRADE

The subgrade may be considered that part of the highway foundation which is not altered except through drainage or frost heave treatment. It differs from the sub-base and base in that the latter are foreign materials transported to the site.

In the study of the subgrade, the first step should consist of making a detailed soil classification survey. If this was made in connection with the study of site, then the only additional soil information necessary will be an occasional detailed investigation at locations as indicated by the original soil survey. For instance, it will be necessary to obtain soundings at culvert or bridge sites. Swamps may need more detailed soundings. A more careful inventory of borrow materials may be necessary. Certain drainage situations may need further investigation in order to determine their extent and possible method of treatment.

If the project involves the surfacing of an oil road, service records will be available locating areas of spring break-up and frost heaves. Such areas must be investigated in order to determine causes before designing proper corrective measures.

Generally speaking traffic requirements dictate the type of highway cross section to be used. Beyond the shoulder lines, however, the characteristics of the cross section are greatly influenced by the soil. The modern trend involves the elimination of unnecessary ditches and usin covered drainage when economically feasible. In low cost road construc tion, however, it is usually necessary to provide for surface drainag by means of open roadside ditches. These should be constructed as shallo as surface drainage requirements will permit and as far from the highwa centerline as practical. The gradient of the ditch is as important as it size. There are too many ditches serving as water reservoirs, rather tha as water conduits.

It is no longer customary to attempt to correct inadequate groun drainage by means of roadside ditches. The accepted method for avoid ing poor ground drainage consists of either constructing a grade o adequate height above water table or, when this is impossible, installin tile edge drains to lower the ground water table and remove seepag waters. Road side ditches used on level sandy soils, such as the Graylin Sand may be built as valley ditches of depths less than one foot. O heavy textured Miami Loam or Selkirk Silt Loam, however, the roadsid ditches should be of sufficient depth to provide drainage for the necessary sub-base. Ditches varying from one and one-half to two feet in dept have been found adequate for this purpose; when right-of-way condition prevent the use of ditches, sub-bases must be drained by means of tile.

In preparing a subgrade which will provide uniform and ample support for surface courses it is important to eliminate conditions which cause winter heaving and frost boils during spring thaws. When the subgrade consists of widely varying soil textures over short distances, either vertically or horizontally, an excellent treatment consists of mixing the subgrade soil using equipment of the scraper type. This equipment excavates in thin layers, deposits at the end of the excavation in thin layers and then brings the soil material back into the excavation in the same manner resulting in an absolutely uniform subgrade soil. This method should be used only when there are sufficient excellent subgrade soils present in the cut being treated to insure a satisfactory final mixture.

When frost heaving textures, such as very fine sand or silt, are encountered in sizable bodies, excavation of the offending material is now universally practiced. Care must be used, however, to see that the back-fill is of a texture similar to the soil materials in the remainder of the area. A sand backfill in an area of sandy clay, has been known to cause winter bumps due to differential heaving at the point of contact between the sand and sandy clay. The depth of treating the subgrade for the prevention of frost heaving varies with the severity of the climate and the nature of the frost heaving deposits encountered. In Lower Michigan depths vary from 2½ to 3 feet and in Upper Michigan from 3 to 4 feet. The width of treatment has long been standardized at 4 feet wider than the proposed metal.

In a discussion of organic soils this paper must be limited to top soils. The subject of peat bogs is sufficiently extensive to necessitate a separate field of study. In the construction of low cost roads the "A" horizons or top soils of the heavier soils have been responsible for many spring failures. In areas of level terrain the usual technique of grade building consists of excavating ditches with a power grader and constructing the embankment out of the resulting fill material. In many cases this practice results in subgrades composed of highly organic top soils characterized by their high moisture holding capacity and their spongy or springy nature. During the spring break-up season these subgrades are subject to severe break-up and it is a common sight to see the black mud pumping up through the metal and flowing over the surface of the roadway. Conditions such as these must be anticipated from the information contained in the soil classification survey. Top soil materials should be used as covering for raw slopes rather than as subgrades for new road surfaces.

SUB-BASE

One of the important problems confronting the highway engineer consists of developing road foundations capable of supplying uniformly adequate support during the entire year, including the spring break-up period. Inasmuch as stability in granular soil textures are less affected by variations in moisture content and temperature than are plastic soils, most highway builders have resorted to the use of sand or gravel as sub-bases for all road metals to be constructed over clay soils.

In the construction of a gravel surface a sand sub-base over a clay soil may not be so important, inasmuch as continuous spring maintenance can keep the road passable for ordinary light traffic. If, on the other hand, the gravel is to later serve as a base for a higher type surfacing which does not permit periodic dragging it becomes important that foundations are such as to prevent wheel loads from breaking through the surface. A gravel surface, being pervious to the movement of capillary moisture, permits more rapid drying than other types of road surfaces thus shortening the period over which the subgrade may be soft and unstable. It behooves the road builder, therefore, to use his various soil materials in such manner as to end up with a foundation capable of supplying year around uniform service if he is to avoid reconstruction and excessive maintenance costs. To accomplish this result the first step consists of classifying the soils along the proposed right-of-way to supply the design engineer with information as to the need for sub-bases, how thick they should be and the location of suitable material for their construction. There are very few sections in the Great Lakes region where suitable soil textures cannot be found within reasonable haul.

Experience indicates that the thickness of base courses can be considerably reduced over sandy subgrades. Obviously, therefore, the cost of sub-base may be offset, at least in part, by the reduction in the cost of base construction. Using costs based on current prices, it was found that approximately 3 inches of sub-base could be placed for the cost of one inch of stabilized gravel base course. By reducing the stabilized base course thickness from 8 to 4 inches, it became possible therefore, to build a sub-base of sand or pit-run gravel 12 inches thick without increasing the cost of construction. In other words, it was possible to construct a 16 inch granular blanket over the clay subgrade for the price of the old 8 inch gravel.

In the construction of sub-bases there are 3 general sections which have been used: The trench, the inverted T and the full section. The trench section consisted of trenching 2 feet wider than the proposed metal and to a depth equal to the depth of the desired sub-base. Sand or pit-run gravel, was then placed in this trench and drained by means of bleeders through the shoulders placed at 50 foot intervals on each side and staggered. Experience found that these bleeders were inadequate to properly drain the subgrade with the result that the so-called continuous bleeder came into use, thus giving rise to the inverted T section. This section consisted of a 12 inch layer of sand below the metal, with 6 inch layers of sand through the shoulders to the ditch bottom. This section permitted good drainage, provided that construction control prevented wheel traffic from squeezing dikes of clay through the 6 inch layer of sand during construction operations. The inverted T section, however, involved too many small construction operations causing a change to the full sand section which is now used exclusively. This section consists of excavating the clay grades to 12 inches below the final earth grade using a ¼ inch per foot slope from center-line to the roadside ditch bottom and then completing the cross section to flat earth grade with sub-base materials.

If the sub-base material is a loose and incoherent sand, an item of of manipulation is included in the contract for the purpose of stabilizing the upper portion of the sub-base. The latter is constructed to a sufficient thickness and stabilized to a sufficient depth to permit trenching for the gravel base course, and at the same time leaving the required thickness of sub-base material below the base course. The purpose of the trenching is to provide stabilized shoulder material needing no further treatment other than compaction. The stabilizing agent does not always consist of clay, but may include loamy top soils or the sticky ''B'' horizon, of such gravelly soils as the Fox or the Bellefontaine series. Theoretically it could consist of chemicals, oils or any product which would increase the stability of a loose sand. Generally speaking, it is desirable to use only sufficient stabilizing agent to permit thorough compaction of the base course without danger of displacing loose and incoherent sub-base material. The preferred sub-base consists of a pit-run gravel free of clay and silt, but sufficiently graded to permit compaction without the addition of stabilizing agents.

In building a sub-base there are a number of construction details which need attention. Trucking over an unstabilized sand base should not be permitted when the underlying clay is soft and subject to rutting. In distributing the stabilizing agent care should be used to see that it does not become concentrated in deep wheel tracks in the sand.

BASE

The most common base course used in Michigan, consists of plant processed stabilized aggregate with specifications as follows:

	Gravel	Crushed Stone
Passing 1″ square screen		100%
Passing ¾″ square screen	100%	80-95
Passing ⅜″ square screen	65-90	60-80
Passing No. 10 sieve	35-50	40-55
Passing No. 40 sieve	20-35	20-35
Passing No. 200 sieve	5-20	5-20

The fraction of the mixture passing No. 40 sieve shall have a plasticity index within the following limits, depending on the treatment which the surface is to receive as specified in the proposal:

Type of Treatment	Plasticity Index
Base for bituminous surface treatment	1-6
Surface for chemical treatment	5-9
Surface for the use without treatment	8-12

In manufacturing base material, the gravel is usually produced during ate fall, winter or early spring and stock piled near the proposed surfacing project for later processing with binder. This processing requires a plant set-up in order to insure uniformity in the finished product. Water and from 5 to 10 lbs. per cu. yd. of chemical is added during the mixing process and the final mixture loaded into trucks and placed on the road while it is still wet. The operation of placing is now often very satisfactorily accomplished using an asphalt paver.

Experience emphasizes a number of facts about the use of gravel bases. If for instance, the road is to be built over heavy soils like the Miami or Ontonagon Series, a thickened base is not a substitute for an adequately designed and properly constructed sub-base. If a relatively impervious bituminous surface is to be placed over a clay stabilized gravel thus slowing up the rate with which the base and subgrade can dry in the spring, it is important that drainage is provided for the base through the medium of an open and porous sub-base. Again, if a base course is placed during freezing weather so that it becomes frozen before it is thoroughly compacted a complete spring break-up is likely to result.

In connection with the compaction of bases, there is a tendency to assume that compaction and structural stability are synonymous and that the base is ready for surfacing as soon as it has been well rolled and has taken on an external appearance of stability. Burggraf* has illustrated with special equipment of his own design, the difference between compaction and stability. He has found that 90% of the ultimate stability develops during the seasoning period and that roller compaction accounts for only 10%. Poor base stability is often reflected in surface failures as soon as the finished road is subjected to traffic.

CONCLUSIONS

1. The spring break-up season is the critical season of the year and foundations must be built to carry traffic loads without failures

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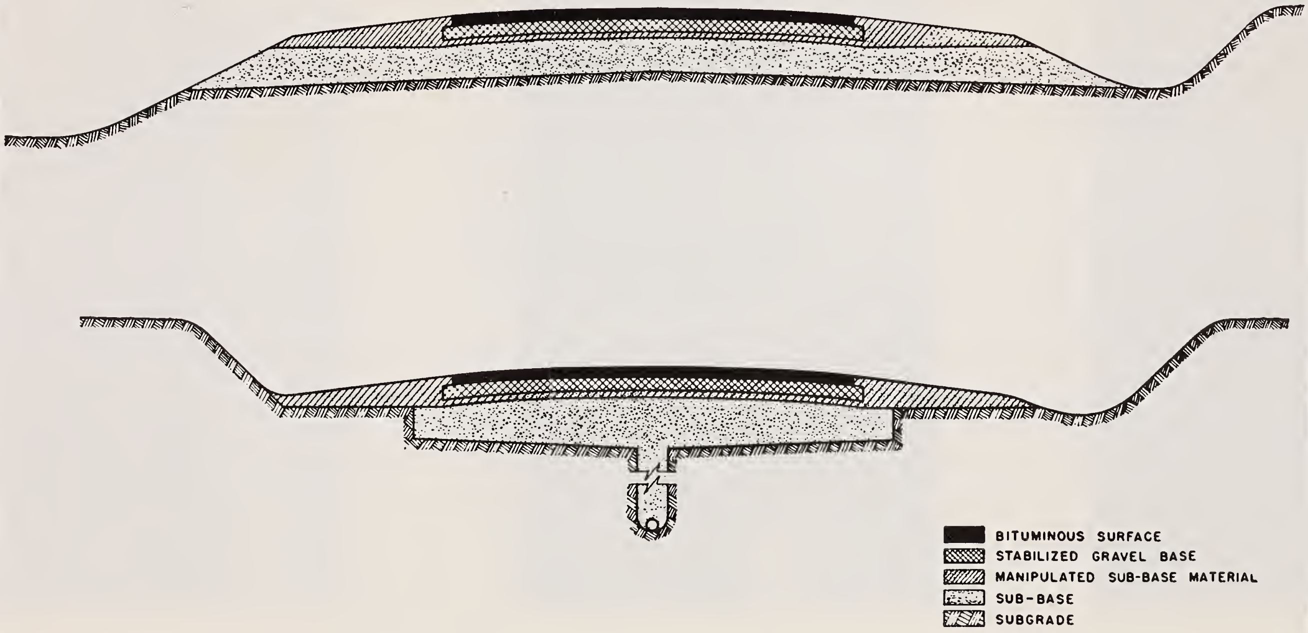
*Fred Burgraf, Research Engineer of the Calcium Chloride Association. May-June, 1939 issue of Calcium Chloride Association News.

in order to avoid expensive reconstruction after this period. Absence of failures in a maintained gravel surface is no guarantee that failures will not occur after bituminous surface treatment is placed.

2. The importance of drainage cannot be over emphasized. The grade line should be located at least 4 feet above normal water table Ordinary roadside ditches should not be relied upon to remove ground water. These are for the removal of surface waters only For this purpose gradient is as important or more important than size if ponding along the road sides is to be prevented.
3. Maintenance records of road behavior over a period of several years are of great help to the designer in preparing plans for reconstruction. Single season records are usually inadequate since seasons vary greatly in the severity with which they affect highway structures.
4. In connection with the construction of bases the importance of curing in order to obtain structural stability should not be overlooked. It is desirable to permit a base to go through one winter under traffic before placing the surfacing.
5. There has been a tendency in the past to assume that only the highest type of highway construction warrants the cost of a soil survey. Low cost road surfaces are more dependent on soil survey information for success than is any other type of surfacing.
6. The present trend in the improvement of highways is to spend a greater part of the road dollar in foundation construction. If the foundation is properly constructed the surface can be made to support an increasing volume of traffic by means of additional low cost surface treatments without extensive base reconstruction.

TYPICAL CROSS-SECTIONS FOR LOW COST BITUMINOUS SURFACED HIGHWAYS


■ BITUMINOUS SURFACE
▨ STABILIZED GRAVEL BASE
▨ MANIPULATED SUB-BASE MATERIAL
▨ SUB-BASE
▨ SUBGRADE

No. 1. Sampling clay to determine suitability as stablizing agent.

No. 2. Spreading the stabilizing agent over the sand sub-base.

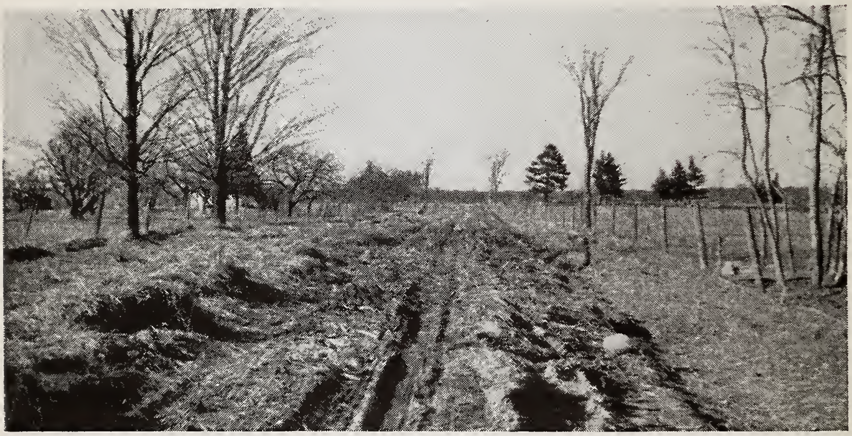
No. 3. Disking the stabilizing agent into upper two inches of sub-base.

No. 4. Trench in stabilized sand sub-base ready to receive base course.

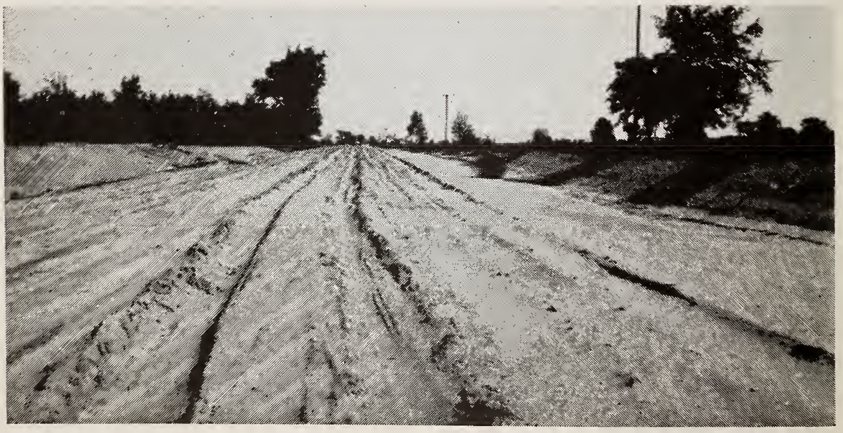
No. 5. Producing stabilized gravel for the base course.

No. 6. Placing the stabilized gravel base course with an oil aggregate spreader.

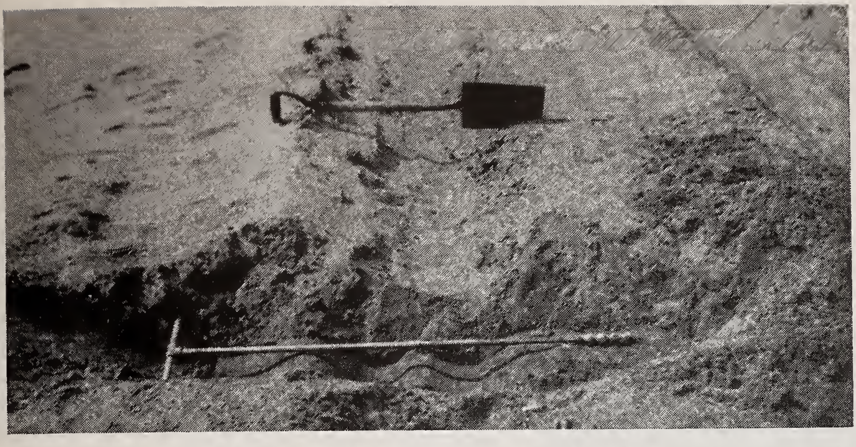
No. 7. Finished base course curing under traffic before bituminous surface is placed.



No. 8. Traffic carried on hay and bark over area of spring break-up in gravel road surface. The use of granular sub-bases offers the most successful cure for this condition.



No. 9. Wheel tracks in sand sub-base must be removed before placing stabilizing agent.



No. 10. Wheel traffic on sand sub-base may cause ruts in underlying soft clay. These ruts interfere with proper sub-base drainage.

Development of Base Construction in Iowa

By

JAMES W. JOHNSON
Laboratory Chief
Iowa State Highway Commission

It is the duty of the engineer responsible for the economical construction of roads to utilize to the fullest extent those natural resources most plentiful in his section of the nation in order to avoid the cost of materials made expensive by long transportation.

The material most plentiful in Iowa, as in most other states, is soil. The highway engineer here is possibly a little more handicapped by the fact that the soil is better suited to agricultural purposes than it is to the construction of highways. However, since soil is the most plentiful and, therefore, our cheapest road building material, we must by careful selection and manipulation use it so as to require a minimum amount of both the base material and the more expensive wearing course.

With this thought in mind, the development of the lower cost of flexible type base has been toward a selection and treatment of the soils in the subgrade, the careful construction of a base of sufficient thickness to transmit the wheel loads to the subgrade, and the application of a bituminous wearing course to withstand the abrasion of traffic.

SOILS OF IOWA

Iowa is situated in the glacial and loessial soil province of the United States and as the name implies, the large part of the surface deposits has originated from deposition by ice or by wind. Alluvial deposits left during the glacial period, or by our present stream action, constitute the major source of our gravel. Beneath the deposits of glacial drift, loess, and alluvial gravels or soils are the stratified rocks which consist of limestone, sandstone, or shale. These rocks also contribute a large share of our coarse aggregate used in road construction.

The agricultural department recognizes five soil sub-provinces within the state, but in our road construction we encounter six major soil areas: three glacial drift sheets and three wind-blown deposits. The Wisconsin drift, the most recent of the glacial deposits, covers the northwestern central part of the state. In this area there are many peat beds or muck deposits which constitute a serious road building problem wherever encountered. The loess deposits which are located in the western, southern, and eastern part of the state, in general cover the glacial sheets, and range in depth from thin layers in the central part to more than 100 feet near the Missouri River.

The physical properties of the various general types of soil are given in Table No. 1.

An examination of the physical characteristics in Table No. 1 shows that many of the soils in all the soil areas must be used with care in the construction of a subgrade for any type of road. The peats or muck, whenever encountered near the surface or when found in any depth, necessitate the removal of the material or relocation of the road. The loess soils, most of which will pass a No. 270 sieve and have a silt content from 50% to 80%, are found in varying depths over the more impervious and better graded drift soils, and because of their high silt content, these soils may be very stable at one water content and become very plastic when the water content is increased only a few per cent. An examination of the table shows that in its undisturbed state, some of the loess soil contains 34.9% of water computed on the dry weight of soil. The dry weight of soil in this particular case was only 84.9 pounds per cubic foot. These figures may not appear startling unless they are analyzed more carefully. One cubic foot of this soil when reduced to an absolute volume basis contains 50.4% of solid material or soil, 2.1% of air, and 47.5% of water or 3.56 gallons per cubic foot of material. Extreme cases of water content in subgrade soils are found in the weathered till of the Kansan Drift, samples of which have been found to contain 4.2 gallons

TABLE 1

RANGE OF PHYSICAL CHARACTERISTICS OF COMMON IOWA SOILS

	Wis. Drift		Iowa Drift		Kan. Drift		Mo. Loess		Miss. Loess		South. Ia. Loess		Peats & Muck	
	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
Gravel	0	24	1	4	0	10	0	0	0	1	0	1	0	4
Sand	9	66	15	56	4	76	1	8	1	10	2	5	12	64
Silt	5	61	17	51	8	66	58	73	61	82	42	63	24	70
Clay	4	38	15	33	7	50	26	33	17	35	30	40	11	20
Colloids	2	22	7	18	4	37	12	20	7	19	14	26	3	15
L. L. L.	19	58	15	43	22	65	32	47	25	42	32	67	48	151
L. P. L.	12	29	12	22	12	24	17	23	16	20	18	23	38	135
P. I.	4	31	3	27	8	45	15	26	8	23	17	46	8	17
Shrinkage Limit	8	26	11	17	10	18	16	20	13	20	11	20	33	66
Shrinkage Ratio	1.55	2.05	1.81	2.01	1.68	2.09	1.75	1.83	1.76	1.87	1.70	2.02	0.72	1.30
*Modified Shrinkage Ratio	1.55	2.05	1.67	2.05	1.66	2.08	1.52	1.74	1.48	1.82	1.63	1.90	0.56	1.39
Bulk Density (Undisturbed)	1.02	1.84	1.24	1.90	1.27	1.97	1.17	1.54	1.20	1.65	1.33	1.63	0.34	0.75
Moisture (natural)	9.0	49.0	12.0	33.0	11.6	31.5	10.4	28.2	14.6	34.9	22.0	31.4	80.0	244.0
Proctor Density	1.81	2.01	1.63	1.94	1.71	1.93	1.41	1.71	1.37	1.84		1.86		
Proctor Moisture	11.0	15.0	11.5	20.0	11.6	19.0	18.1	26.3	14.6	25.5	13.9			
U.S.B.P.R. Sub. Group	A-2,3, 4,6		A-2,3, 4,6,7		A-4,6,7		A-4,7		A-4,7		A-6,7		A-8	

*Shrinkage Ratio Including Material Larger Than No. 40 Screen.

per cubic foot of material. The water in peat soils may be as high as 6 gallons per cubic foot of material. These figures are not mentioned with the thought of conveying the idea that the Iowa soils are the only ones which present problems that must be given serious consideration when constructing a subgrade for any type of base, but rather to show the need for careful study of the soil by a soil survey party.

SOIL SURVEYS

It is the duty of our chief of soil surveys to analyze the problems presented by every grading job or proposed base project to determine the extent of the soil survey necessary to make the proper recommendations to the design department for soil treatment. Such soil surveys are extensive enough to make a soil profile over the entire project and to make test borings in all ''cut sections.'' Test borings are made to a depth of four feet below the proposed grade line and all such borings include, in addition to a record of soil type and samples of soil, the taking of undisturbed core samples in at least one location two feet below the proposed grade line. Core samples have been taken to depths of 35 feet. Density and moisture tests are made on these core samples and with this information and the laboratory tests on soil samples, the subgrade condition may be determined and a treatment decided upon.

Soil surveys on existing graded roads or graded roads proposed for base construction are also made by the soil party. A careful record kept by the maintenance department is helpful in planning the survey of this type. The extent of the ''break-ups'' or ''frost-boils'' is recorded by the maintenance crew and stakes set on the edge of the right-of-way give a permanent and easily located record of the extent of soft areas which have occurred in the spring.

The subgrade treatment recommended by the chief of soil survey party may include any one of the following types:

1. Porous type frost boil treatment.
2. Soil substitution.
3. Subgrade removal, drying, and recompacting.
4. Thickened base.
5. Drainage by means of deepening ditches or placing intercepting drains.

The treatments are all self-explanatory with the possible exception of the porous type treatment. This treatment consists of constructing a porous layer of mineral aggregate in a trench excavated in the existing road bed, covering this porous layer with selected material, reshaping the road, and providing drains for the porous layer as provided on the plans.

A type of instrument new to Iowa has been placed in service by our soil survey party and is intended to measure the resistance of the subgrade to lateral compression or displacement.

The instrument is shown complete in Figure No. 1 and consists of a pressure reservoir for supplying, without pulsations, liquid pressure to the penetrometer. A gauge records the load. The penetrometer in its present design operates in an 8-inch auger hole, but it is planned to make the instrument smaller so that it will work in a 6-inch auger hole, which is the size of hole now used in making soil surveys. Figure No. 2 shows the apparatus in place in an 8-inch hole and this picture with Figure No. 1 shows the shape of plunger used. The penetrometer is approximately three inches wide and six inches high, and is so designed that the amount of penetration into the subgrade is the same at all points. It is possible that other shapes may be better than the present design. The small springs are employed to retract the plunger after the load has been applied so that the instrument may be removed from the hole after the test is made.

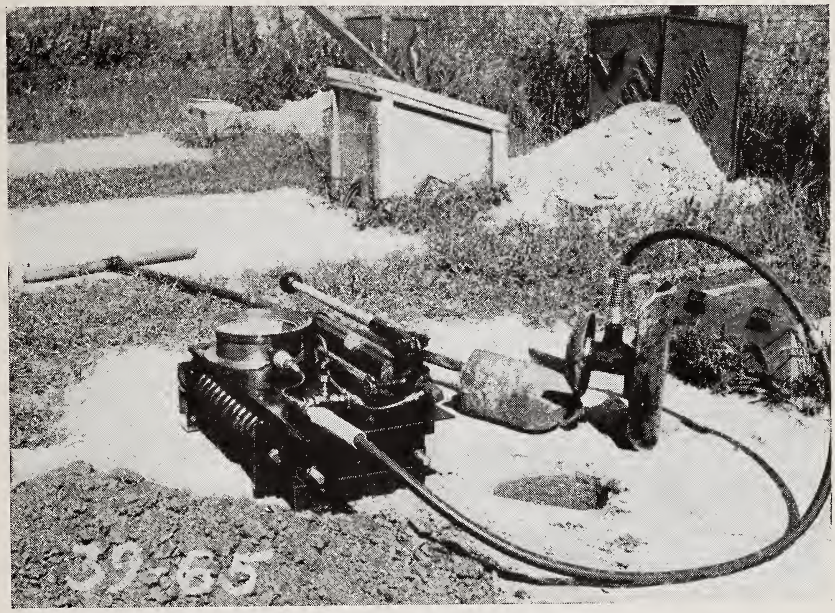


Fig. No. 1. Apparatus Complete

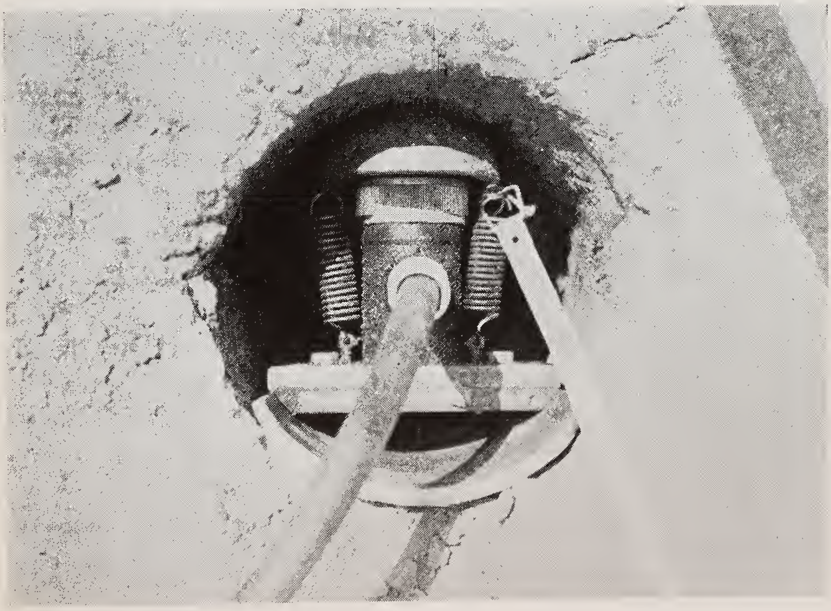


Fig. No. 2. Penetrometer in 8″ Auger Hole.

An example of the results obtained with this instrument on the same type of soil, when the only variable was density and water content, is given:

	Soil Constants — Per Cent						Penetrometer
Test No.	Silt	Clay	Colloids	P. L.	Solids	Water	Load—Lbs.
1	72	26	17	21	57	26	800
2	72	26	17	21	66	17	3400

No claims are made for the instrument but it is hoped that data may be collected by an examination of failed areas, areas which show signs of approaching failure, and locations where no failures have occurred, so that a numerical value may be obtained for the subgrade resistance necessary to hold up various types of bases or surfaces. It is believed that such a numerical value will convey to the engineer who is not familiar with the numerous soil constants a better idea of the subgrade condition than we are now able to give him.

BASE CONSTRUCTION

The first soil-aggregate bases were constructed in Iowa in 1934. The specifications for these first bases were rather broad but it was intended that the final mixture of coarse aggregate and soil binder should comply closely with the grading limits recommended by the Calcium Chloride Association at that time. Any soil which had a plasticity index of 20 or more was considered a good soil binder.

The soil was pulverized on the road by any means which would reduce the material in size so that 95% would pass the No. 4 screen. The soil and gravel were then mixed together by means of blade maintainers, water added, and compaction accomplished by using either heavy flat rollers or pneumatic-tired rollers. The degree of compaction was not specified, but rather was left to the judgment of the engineer and the ''heel'' or ''screw-driver'' methods were a common means of determining the amount of rolling required.

This general method of construction was followed through 1936, with changes in the requirements for grading of material and construction procedure being made from time to time to improve the quality of the finished base.

The compacted thickness of our early bases was not specified. The quantity of gravel was usually 1,000 cubic yards per mile and this gravel with the binder clay would give a base thickness of about three inches. The quantity of gravel was increased to 1,600 yards per mile in 1935 and this gave a compacted base about 4 inches thick. In 1937 the quantity of material was specified in tons and the amount was such as to give a compacted base thickness of 5 inches. This thickness has been standard since that time except in places where base thickening is employed over weak subgrade areas.

Two types of base courses are commonly used in Iowa at the present time: (1) rolled stone, and (2) soil-aggregate batch mix.

The "rolled stone" base course consists of a uniform mixture of coarse and fine particles of crushed limestone, crushed by any means the contractor elects to use to produce the grading specified. This type of base was designed to be used in that part of the state where gravel is scarce and the limestone beds are separated by shale seams, making the stone unsuited for concrete aggregate. The shale seams impart to the crushed limestone sufficient binding properties so that the material compacts to a dense base.

The "soil-aggregate" base is specified as a uniform mixture of gravel or crushed stone plus the necessary fine materials such as binder soil or sand, or both, to produce a finished mix as specified. In a few cases it has been possible to find a natural gravel which would comply with our specifications for a soil-aggregate base. It is usually necessary to add from 10% to 20% of binder soil to produce a mix of this type. When crushed stone is used as a coarse aggregate it is sometimes necessary to use a sand in addition to the binder soil to secure the required grading.

The specifications for the two types of bases are very similar and an outline of the requirements is given below:

Grading — Final Mixture

Sieve No.	Per Cent Passing
1½"	100 Minimum
1"	85 to 100
¾"	75 to 100
⅜"	60 to 95
No. 4	45 to 85
No. 10	35 to 70
No. 40	*20 to 35
**No. 100	12 to 28
No. 200	9 to 22

*Note: 15% for rolled stone base.
**Note: Not specified for rolled stone base.

The fraction passing the No. 200 sieve shall not be more than ⅔ the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve shall have a liquid limit not greater than 25.

Plasticity Index

	Rolled Stone	Soil-Aggregate
State Projects	0 to 12	4 to 12
Federal Aid Projects	0 to 6	3 to 6

The grading limits were arrived at partly because such limits would allow the use of a large part of our natural gravels without extensive production processing, but largely because a great number of laboratory density determinations showed that a rather wide range in the coarser sizes did not affect the density of the resulting base so long as the grading of the material below the No. 40 sieve was carefully controlled.

The present practice in designing a soil-aggregate mix is to make laboratory Proctor tests using various amounts of soil in order to specify a soil content which will give maximum density. Since our soil-aggregate mixes are all batch-mix type and we require weighing of the materials to within 5 pounds, it is possible to specify close control of the aggregate used.

The mixer used by the contractor is usually a concrete mixer, previously used in paving work, or a mixer of the "Blythstone" type.

Other general phases of our specifications require that the subgrade be scarified to a depth of 6 inches, shaped, and recompacted to 95% of Proctor density. It is required that the base material be compacted to Proctor density or until the per cent of solid material is 81 or greater.

The mixed material is spread on the road using various types of spreader boxes or blades, and compaction is accomplished in a number of ways depending upon the equipment possessed by the contractor and upon his idea how the job should be done. Some bases are compacted by a sheepsfoot roller, while other bases are consolidated in two or more lifts, each of which is rolled with flat or pneumatic-tired rollers. Other compacting equipment developed by one contractor consists of a variety of rollers used in connection with tampers and vibrating compactors. A construction procedure showing the complete process of a base job is shown in picture form, Figures 3 to 11, inclusive. When this job was started, the mixed material was placed with the spreader boxes and given a preliminary rolling with a light roller. Compaction was accomplished by the use of three vibrating tampers. Final compacting and smoothing was given with the "Roll-a-Plane." The average compaction obtained by this early method is given below:

	Per Cent of Proctor Density 130.5 Lb./Cu. Ft.		
Compaction Treatment	Top	Center	Bottom
Longitudinal Roller	99.5	97.5	92.0
One Vibrating Tamper	99.8	98.5	96.0
Two Vibrating Tampers	100.0	99.5	98.0
Three Vibrating Tampers	100.5	100.0	99.0
Roll-a-Plane	105.0	102.5	99.5
Moisture after Roll-a-Plane (%)	6.6	6.4	8.0

Density tests on the later method of base construction show an average of 104% of Proctor density and a density in the top 2¾ inches of 109.5% of Proctor density or a dry weight per cubic foot of 142.9 pounds.

This same contractor is constructing a 7-mile soil-aggregate base job at the present time and the strike-off and tamping machine shown in Figures 6 and 7 are equipped with tampers covering the full width of base instead of only 20 inches on each edge. The tampers are also located at an angle so that the entire surface is compacted. The vibrating compactors are not used on this project. The progress being made by this contractor is 1,400 feet per 10-hour day. Another contractor using two Koehring 27-E pavers for mixing the materials and consolidating in one layer with sheepsfoot rollers and pneumatic-tired rollers, made an average progress of 2,500 feet per 10-hour day on an 8-mile project.

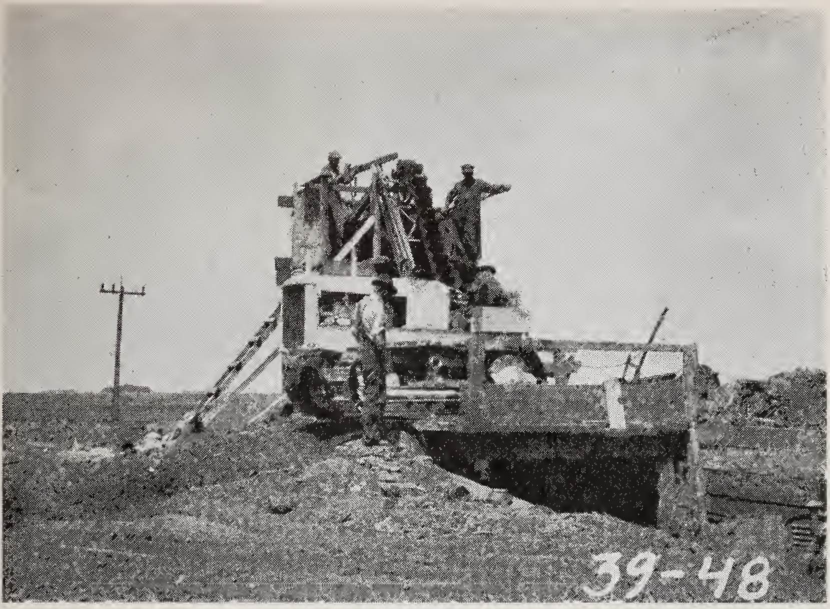


Fig. No. 3. Pulverizing Binder Soil. All soil is made to pass a No. 4 screen.

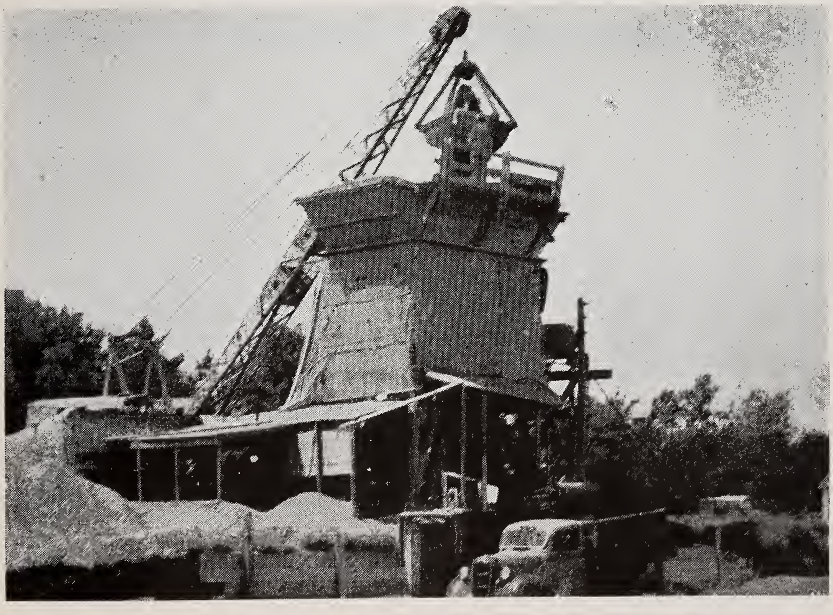


Fig. No. 4. Proportioning Plant. Two-yard Blythstone mixer in center of picture with conveyor belt to carry mixed material into the trucks.

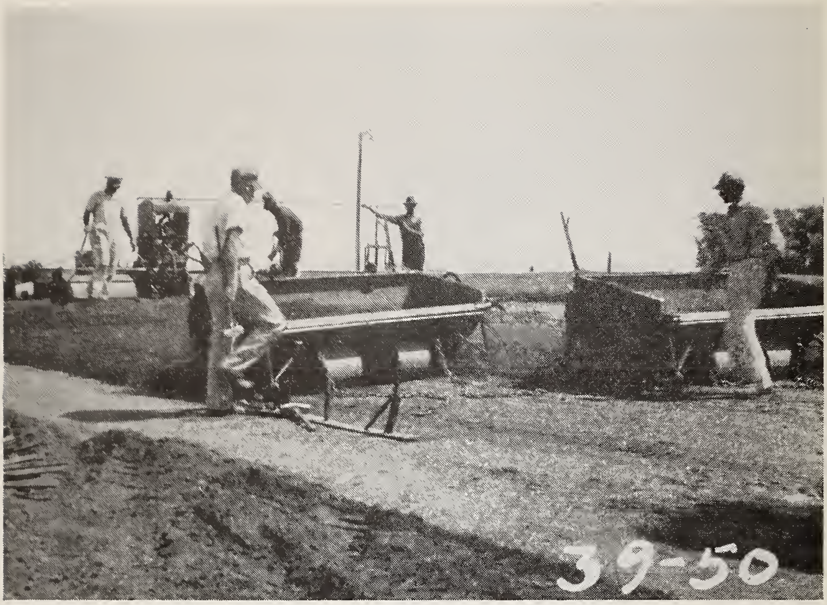


Fig. No. 5. Spreader Boxes for Spreading the Mixed Material.

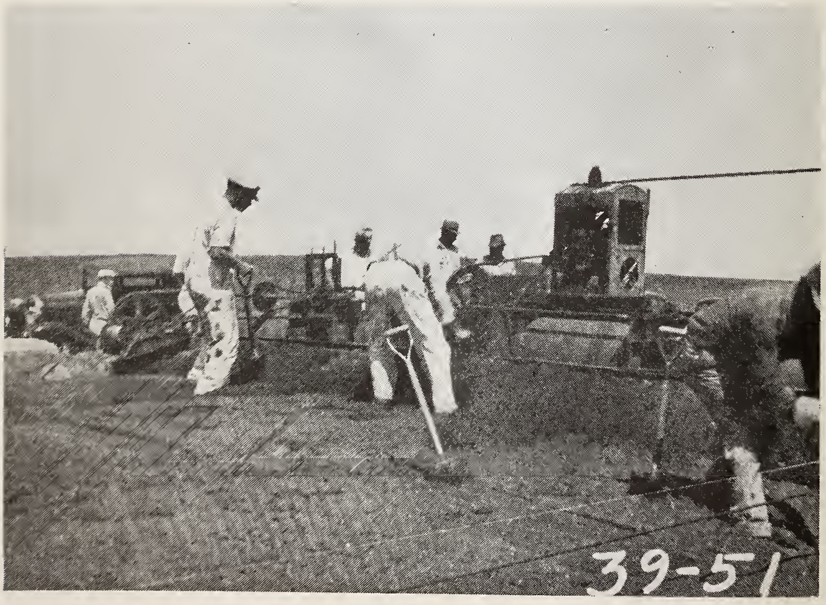


Fig. No. 6. Strike-off Machine-Edge Tamping and Preliminary Rolling.

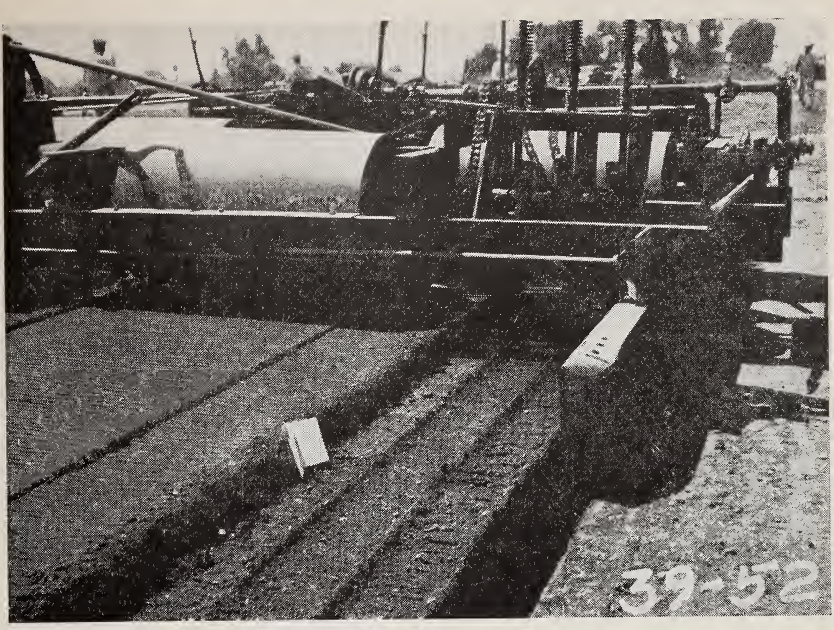


Fig. No. 7. Rear View of Machine Shown Above—Edge Tampers Compact Base Material About 3½″ More Than Rollers.

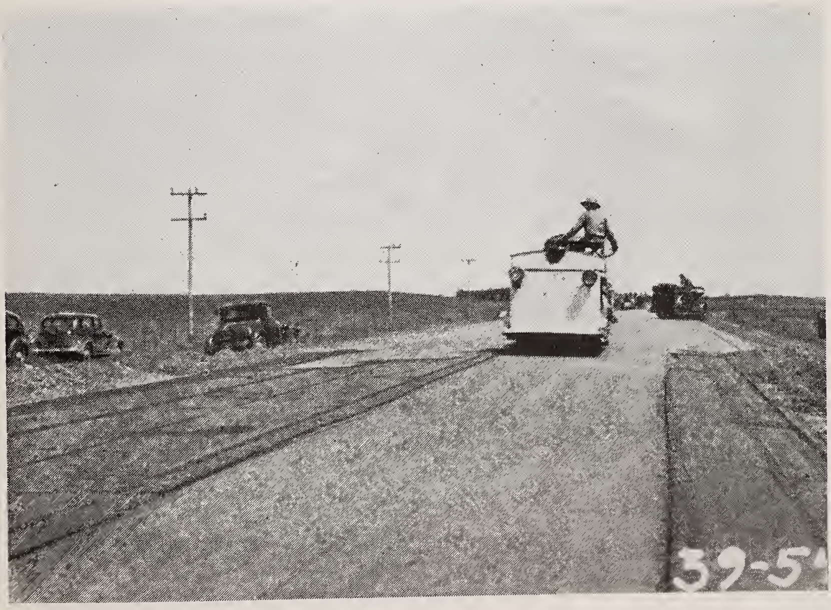


Fig. No. 8. Longitudinal Roller (8-ton) Behind Strike-off Machine. In the background the shoulder material has been placed and given initial compaction with longitudinal roller. The diagonal roller (5-ton) is seen in the background.

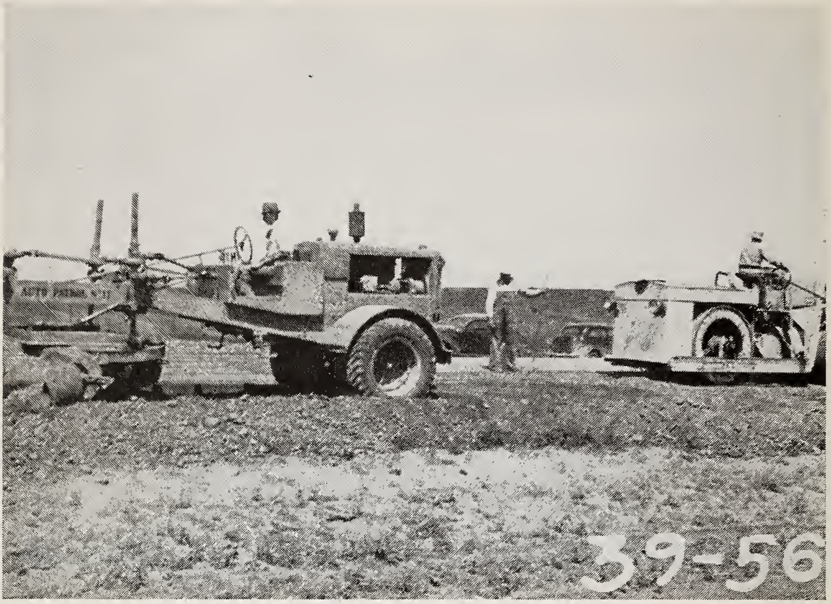


Fig. No. 9. Auto-Patrol Spreading Shoulder Fill.

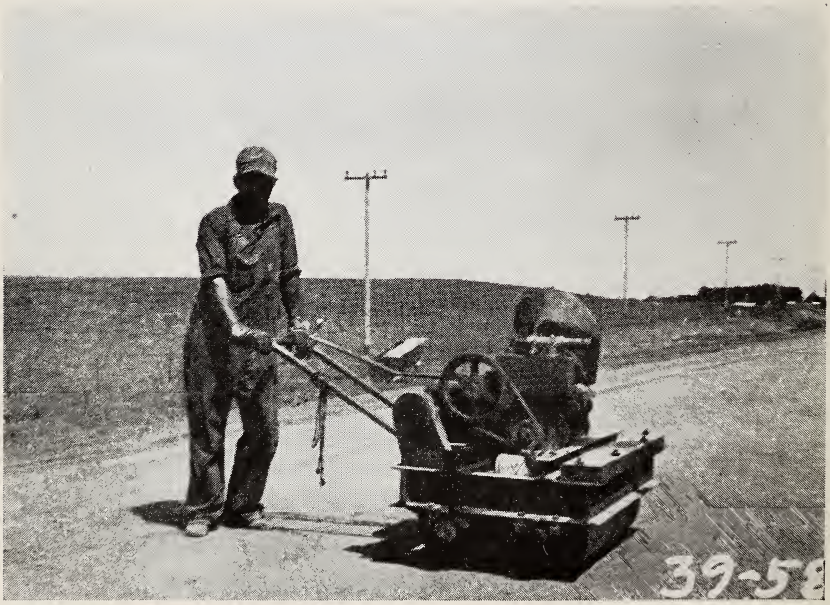


Fig. No. 10. Vibrating Compactor, Self-Propelled. Used back of diagonal roller.

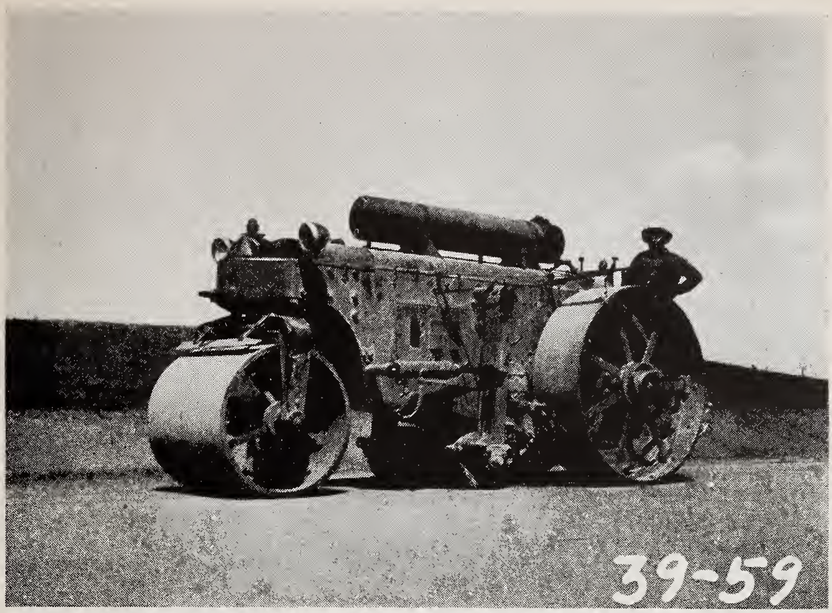


Fig. No. 11. Roll-a-Plane Giving Final Compaction and Possible Aid to Riding Surface (15-ton Roll-a-Plane).

EXPERIMENTAL BASES

Other types of bases which might be considered experimental that have been placed, are: (1) bituminous treated earth; (2) emulsion stabilized sand-soil; (3) emulsion mat base 5 inches thick; (4) soil-aggregate base 5 inches thick—2-inch emulsion mat; and (5) soil-cement.

The project which was longest and upon which the most complete data were kept was the bituminous treated earth base. This project was constructed in the loessial area of the state and was divided into two sections: One 5.962 miles long, and the other 7.408 miles long. The only difference in the two sections was in the grading of the soil; the longer project contained about 600 cubic yards more limestone surfacing material per mile than did the other section. The physical properties of the soil in the two sections were as follows:

Section No.	Gravel	Sand	Silt	Clay	Colloids	L. L.	P. I.
1	8	10	52	30	14	39	19
2	34	15	32	19	10	37	17

The bituminous materials used to stabilize the two sections were MC-3, SC-3, SC-5, and emulsion. Other sections contained TC-2 and cement, and there were soil-aggregate and soil-aggregate calcium chloride sections. All mixing was done with a Woods traveling mixer with the exception of the soil-cement sections.

The following data have been taken on this project.

1. Moisture content from time of mixing.
2. Density of all sections from time of accepted compaction to present date.

3. Subgrade moisture and density.
4. Hveem stabilometer tests on cores taken from base.
5. Horizontal shear tests on base and subgrade.

A record of the moisture content of the sections at the time of mixing and at stages during the construction is shown in Table 2. All the treatments with the exception of the cement sections were placed in 1937. The road was primed and allowed to go through the winter before the wearing course was applied. The cement sections were placed early in 1938 and a bituminous wearing course of the inverted penetration type, consisting of 0.6-gallon of SC-7 or TH-2 and 60 pounds of aggregate per square yard placed in two applications, was placed over the entire project. The moisture and density tests made in June, 1938, were taken just before the wearing course was placed.

The densities and moisture contents taken at intervals are shown in Table 3. It is planned to continue these same tests for a number of years.

The average moisture and density of the subgrade immediately below the base is shown below:

	Section 1		Section 2	
Depth Below Base	Per Cent Water	Dry Wt. Per Cu. Ft.	Per Cent Water	Dry Wt. Per Cu. Ft.
First Inch	17.8	1.67	16.4	1.75
Second Inch	19.1	1.65	17.6	1.67
Third Inch	19.2	1.64	18.8	1.65
Fourth Inch	19.3	1.62	18.2	1.60
Fifth & Sixth Inch	19.5	1.59	19.3	1.58

The subgrade is a loess soil containing about 65% of silt, 33% of clay, and 19% of colloids.

Other phases of our investigation consisted in making stability tests using a Hveem stabilometer on 4-inch cores taken from the base, and making "horizontal shear" tests on the base and subgrade immediately below the base. It was impossible to obtain samples for the Hveem stability tests from the soil-aggregate and soil-cement sections.

The results of the Hveem stability tests on the bituminous stabilized sections are shown in Table 4.

The Horizontal shear tests were made on the base and subgrade using a method similar to that first used by Mr. Fred Burggraff, Materials Engineer for the Calcium Chloride Association. The device is shown in Figure 12. It will be noted that the pressure reservoir is the same as that described for making subgrade resistance measurements. The shear plate is flat and has the shape of a parabola, the equation of which is $X^2=5.68\,Y$. The depth of the plate used was 3⅜ inches and the width was approximately 8.80 inches.

The test was made by digging in the base a trench of such size as to accommodate the instrument. The rear bearing plate was enough larger

TABLE 2

MOISTURE CONTENT AT VARIOUS AGES AND DURING PROGRESS OF CONSTRUCTION

				Per Cent of Water at Various Ages			Compaction Started		Compaction Completed		Prime Applied		% Proctor Water
Section No.	Stabilizing Material	Per Cent Stabilizer	When Mixed	10 Days	20 Days	30 Days	Age-Days	% Water	Age-Days	% Water	Age-Days	% Water	When Primed
	MC-3	3.4	30.2	20.5	18.5		6	23.8	23	19.0	24	19.1	97.0
	SC-3	4.2	29.9	22.0	18.8	17.4	7	21.4	35	18.9	35	18.9	102.0
Section	SC-5	4.2	33.4	24.7	21.4	18.8	6	23.6	34	17.2	34	17.2	92.0
No. 1	Emulsion	9.0	31.7	30.3	20.2		16	29.5	28	19.3	28	19.3	97.0
	Emulsion	6.6	34.5	33.3	23.9		16	32.5	28	23.2	28	23.2	113.0
	TC-2	3.2	32.7	18.9			3	27.1	16	18.2	16	18.2	92.0
FN-447,	Cement (% by Vol.)	10.0	15.1	12.7			0	15.1	1	14.5	6	12.7	126.0
Cass	Cement (% by Vol.)	12.0	14.3	13.5			0	14.3	1	14.1	13	13.5	102.0
	Soil-Aggregate		13.0	7.6	6.0		3	10.1	20	6.0	20	6.0	65.0
	Soil-Agg. + $CaCl_2$	0.5	13.0	7.2	5.9		3	10.7	20	5.9	21	6.0	65.0
	MC-3	3.0	20.5	10.6			5	15.1	12	9.3	18	10.3	90.0
	SC-3	3.3	20.0	9.4			5	13.6	12	10.2	16	10.2	90.0
Section	SC-5	3.3	19.7	11.1			4	12.7	12	11.0	16	10.7	94.0
No. 2	Emulsion	6.5	20.7	14.2	13.1		2	22.7	14	14.3	25	13.4	108.0
	Emulsion	8.0	20.6	11.2			2	18.0	10	11.2	14		
	TC-2	2.9	22.0	13.5	11.0		7	15.9	19	11.3	27	10.0	88.0
FN-594	Cement (% by Vol.)	8.0	14.0				0	14.0	1	13.9	2	13.9	119.0
Cass-	Cement (% by Vol.)	12.0	16.8				0	16.8	1	16.7	2	16.7	104.0
Pott.	Soil-Aggregate		14.4	8.7	6.2	5.3	1	12.5	17	5.2	35	5.1	60.0
	Soil-Agg. + $CaCl_2$	0.5	13.4	8.9	6.3	5.7	2	12.4	16	4.8	35	5.6	64.0

Note: Water is expressed as a per cent of the dry weight of soil.

TABLE 3
CHANGE IN MOISTURE AND DENSITY AT VARIOUS AGES

Type of Base Treatment	Oct. & Nov., 1937		March, 1938		June, 1938		Nov. & Dec., 1938		May, 1939		Proctor Tests	
	% Water	Dry Wt./ Cu. Ft.	% Water	Dry Wt./ Cu. Ft.	% Water	Dry Wt./ Cu. Ft.	% Water	Dry Wt./ Cu. Ft.	% Water	Dry Wt./ Cu. Ft.	% Water	Dry Wt./ Cu. Ft.
SECTION 1												
MC-3	19.1	104	14.1	111	13.8	113	14.4	106	14.1	113	19.7	102
SC-3	13.1	104	16.0	109	13.6	112	14.2	106	13.6	114	18.5	104
SC-5	17.2	104	13.8	113	13.8	111	14.1	109	13.3	111	18.7	104
Emulsion (9%)	19.3	102	15.5	106	14.7	111	16.2	106	14.4	110	19.9	99
Emulsion (6.6%)	23.2	93	18.3	103	17.3	109	18.6	102	17.1	107	20.6	103
TC-2	18.2	104	15.5	116	14.0	112	15.8	108	14.8	111	19.8	103
Cement (10%)					15.1	104	14.8	107	13.5	106	9.4	118
Cement (12%)					14.3	101	14.4	103	13.5	106	14.0	116
Soil-Aggregate	6.0	135	6.5	135	5.5	138	6.2	142	5.6	140	9.3	132
Soil-Agg. + CaCl$_2$	6.0	137	5.7	137	4.7	141	5.7	143	5.4	143	9.3	132
SECTION 2												
MC-3	9.3	125	9.3	121	7.8	124	8.8	121	9.0	121	11.4	118
SC-3	9.7	123	10.0	119	8.0	122	9.0	116	9.8	120	11.4	119
SC-5	10.5	123	9.6	119	8.7	119	9.0	116	8.9	121	11.4	118
Emulsion (6.5%)	12.1	119	11.6	115	10.5	117	11.5	114	11.6	117	12.4	115
Emulsion (8%)	13.0	117	12.3	117	8.9	117	10.2	115	9.8	120	12.4	115
TC-2	11.2	121	10.7	119	9.4	121	11.7	115	9.7	118	11.4	118
Cement (8%)					13.9	107	13.1	104	11.6	106	12.3	120
Cement (12%)					16.7	98	13.9	101	13.1	99	15.9	111
Soil-Aggregate	5.7	141	5.6	136	4.4	138	5.8	138	5.4	139	8.5	129
Soil-Agg. + CaCl$_2$	6.2	140	5.6	137	5.2	136	6.1	135	5.6	139	8.7	129

TABLE 4

HVEEM STABILOMETER TESTS ON CORES TAKEN FROM ROAD

Results Reported Are Average

	VERTICAL LOAD — POUNDS PER SQUARE INCH											
	SECTION NO. 1						SECTION NO. 2					
Hveem Gauge Reading	MC-3	SC-3	SC-5	9% Emul.	6.6% Emul.	TC-2	MC-3	SC-3	SC-5	6.5% Emul.	8.0% Emul.	TC-2
Number of Tests	4	4	4	4	4	4	4	4	4	3	1	2
20	90	70	80	70	70	115	70	75	85	70	80	65
40	125	100	115	100	95	170	110	120	125	105	115	110
60	160	130	150	120	120	205	135	155	160	130	140	145
80	185	155	175	145	140	235	165	190	190	160	165	180
100	210	175	200	170	165	270	190	220	215	180	190	210
120	235	200	225	190	185	300	220	245	240	205	215	240
140	260	220	245	210	205	330	245	270	265	225	235	265
160	285	240	270	230	225	365	270	300	295	250	260	295
180	310	265	290	255	245	400	295	325	320	270	280	325
200	335	285	315	275	265	430	320	350	345	290	305	355
% Water	13.8	13.6	13.8	14.7	17.3	14.0	7.8	8.0	8.6	10.5	8.9	9.4
Weight	113	112	111	111	109	112	124	122	119	117	117	121

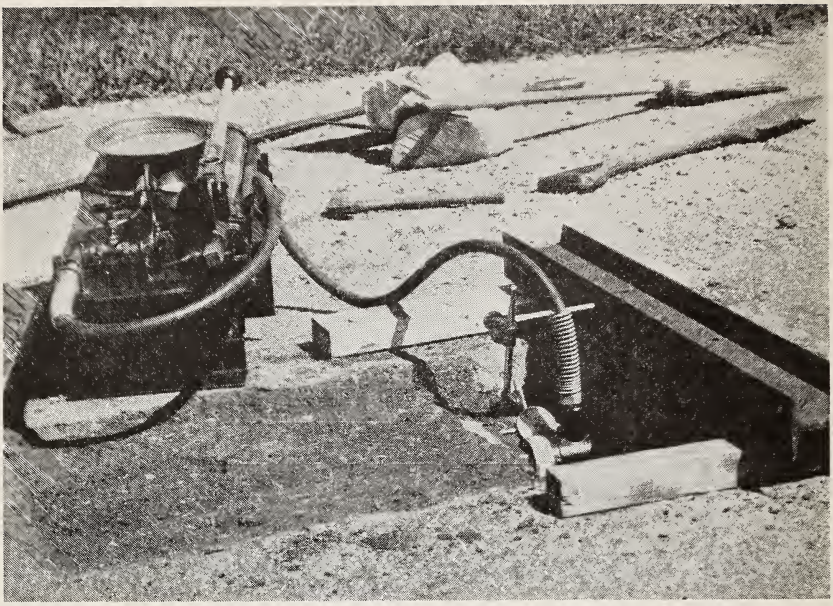


Fig. 12. Horizontal Shear Apparatus. Instrument in place, wearing course removed. Ready to make shear test.

than the shearing plate so that there was little movement in that direction. Fine, dry sand was sifted around both end plates to secure good bearing and the shear test was made by applying pressure at a uniform rate and recording the load for each 0.5-inch deformation until failure occurred. The density and moisture contents of the base and subgrade were determined on the material removed for the placement of the instrument.

The results of the horizontal shear tests are shown in Table 5. Shear tests on representative sections of the subgrade are also shown. The number of tests made was not sufficient to draw definite conclusions on the shearing resistance, but they give an indication of what may be expected of this type of road in this soil area.

An attempt was made to measure the area of the shear plane but in some cases the base was somewhat plastic and the shear plane was not definitely defined. In the more brittle sections which contained less water, a failure would sometimes develop in the form of a cone in front of the shear plate and cause a large area to spall. The opinion was reached that the best way to report results was to tabulate the total load on a certain size shear plate.

BITUMINOUS WEARING COURSE

The wearing course most commonly used in Iowa is approximately ¾-inch thick and is the inverted penetration type, consisting of a prime of 0.2 to 0.4-gallon per square yard, 0.6-gallon of binder bitumen, and 60 pounds of aggregate applied in two applications. This type of wearing course has been adopted for economical reasons following our belief that a base properly constructed on a well compacted subgrade requires a wearing course only thick enough to withstand traffic abrasion and to waterproof the base. Additional economy of this type of mat lies in

TABLE 5

HORIZONTAL SHEAR TESTS

Results Reported as Total Load in Pounds on a Parabolic Shear Plate Having an Area of 20 Square Inches

	TREATED BASE								SUBGRADE			
	Section No. 1				Section No. 2				Section No. 1			
Type of Base Treatment	Total Load-lb.	Dry Wt. Per. Cu. Ft.	% Water	Movement @ Max. Ld. (In.)	Total Load-lb.	Dry Wt. Per Cu. Ft.	% Water	Movement @ Max. Ld. (In.)	Total Load-lb.	Dry Wt. Per Cu.Ft.	% Water.	Movement @ Max. Ld. (In.)
MC-3	2525	114	12.1	0.25	1650	120	8.7	0.10	1550	102.3	14.5	0.15
SC-3	1850	116	12.5	0.20	1240	120	9.9	0.10	1575	106.1	15.1	0.30
SC-5	1950	110	12.1	0.15	1240	122	8.5	0.10				
Emulsion (9%)	1600	115	12.4	0.30								
Emulsion (6.6%)	1950	107	17.6	0.50					1450	96.7	24.4	0.40
Emulsion (8.0%)					1420	120	9.8	0.15				
TC-2	2300	111	14.4	0.15	2220	125	7.3	0.10				
Cement (8%)					6040	110	11.3	0.05				
Cement (10%)	9250	104	14.7									
	8700	107	12.3	0.05						117.3	12.9	
Cement (12%)	6830	108	11.0	0.10	8700	101	11.1	0.07	1950	116.7	13.4	0.30
Soil-Aggregate	1750	139	5.7	0.10	2130	140	5.5	0.10	1650	112.3	15.6	0.20
Soil-Agg. + CaCl^2	1850	143	5.3	0.10	2400	139	5.6	0.10	1650	104.2	18.3	0.20

the fact that our natural gravels lend themselves to this type of construction with little processing. Also, our climatic conditions are such that we seldom have long, dry periods and the aggregate can be placed when it contains some moisture.

Our specification for the grading of the aggregate used in this typ of wearing course is as follows:

	Per Cent Passing			
Sieve No.	Crushed Stone		Gravel	
	Minimum	Maximum	Minimum	Maximum
¾″	100		100	
No. 4	30	50	40	60
No. 8	10	25	20	45
No. 30		10		25
No. 200		8		10

The quality of the material is also specified.

RESEARCH WORK IN PROGRESS

As previously stated, we are keeping a continuous check of the moisture and density of all our various types of bases whether they are experimental or our regular bases. It is the belief of some engineers that the plasticity index of the material passing the No. 40 screen, in soil-aggregate bases, must be kept low in order that a base will not become unstable due to moisture absorption when covered with an impervious wearing course. With this thought in mind moisture tests have been taken at time intervals on representative bases. Typical results are as shown:

		PER CENT MOISTURE					
		Sampled July, 1936	Sampled April, 1937		Sampled Nov., 1938		
Year Built	P. I.	Base	Base	1st Inch Subgrade	Top Base	Bot. Base	1st Inch Subgrade
1934	9.0	5.7	6.3	14.6	5.4	6.1	13.3
1937	8.6				5.6	5.8	18.0
1934	9.3	6.8	7.5	11.6	7.8	7.4	12.5

The same information is shown in Table 2. The plasticity index of the soil-aggregate base in Section 1 is about 8, and about 11 in Section 2.

A number of bases having a P.I. of about 4 are being placed this year and the same information will be taken on these bases.

A research project is in progress in our laboratory, carried on under the title: "Accelerated Oxidation Tests on SC Road Oils." The work in its present form consists of subjection of 100-gram samples of SC oils, both "positive" and "negative," to oxidation by bubbling 140° F. air, at a controlled rate, through the sample which is maintained at 300° F. The duration of the test is 10 hours and any loss in oily constituents during the oxidation process is noted and the subsequent analysis corrected. The original road oil and the oxidized sample are analyzed for ashaltenes and oily constituents. The amount of resins are not determined irectly but are calculated by differences.

The change in the ratio of asphaltenes to resins from the original il to that of the oxidized oil is used to calculate a value designated as he "Oxidation-Index Number."

The oils used in the experiment are those which have been used in onstruction work and a study will be made between laboratory results nd behavior in the field.

The Iowa Engineering Experiment Station is conducting an investigation on "The Distribution of Wheel Loads Through Flexible Pavenents." The project was outlined and is being carried on under the upervision of Mr. M. G. Spangler, Research Associate Professor, Iowa State College. The results of Mr. Spangler's preliminary work may best e shown by giving a synopsis of a paper presented by him at the Eigheenth Annual Meeting of the Highway Research Board, held in Decemer, 1938. This synopsis follows:

"This report describes a type of apparatus for measuring the pressure distribution under static wheel loads on flexible pavements which gives promise of making possible an extensive study of the effects of various characteristics of flexible pavements and subgrades upon pressure distribution.

"The apparatus consists of a series of stainless steel ribbons arranged to slide between thin stainless steel strips. These ribbon assemblies are placed in the plane between the flexible pavement and the subgrade, and are pulled horizontally to measure the total pressure on the ribbons. Then the ribbons are shortened by decrements of one inch and the pull measured for each length of ribbon. The difference between these pulls is proportional to the difference in pressure on the ribbons at the various lengths, and with the coefficient of friction determined, provides a mechanical differentiation of the pressure.

"Observations have been made on flexible pavements in which all characteristics were held constant except the thickness, which varied between 3 and 15 inches. Loads were applied through a pneumatic tire. The maximum pressure on the subgrade occurred directly beneath the applied load and distributed laterally in an elongated bell shape. The maximum unit pressures accompanying a 3,000 pound load were: 65 lbs. per sq. in. at 3 in. thick, 35 and 44 lbs. at 5 in., 24 lbs. at 7½ in., 18.5 lbs. at 10 in., and 13 lbs. at 15 in. thick."

Mr. Spangler's work has created much stimulating discussion of the flexible base problem and he is continuing this study, having the following objectives in mind:

A. Definite knowledge of the distribution of pressure on a subgrade when a wheel load is applied to a flexible pavement surface as influenced by:
    1. The type of base and wearing courses, such as stabilized gravel, sand-clay, soil-cement, crushed stone, etc., with or without a bituminous bound wearing surface.

2. The thickness of the base course.
3. Moisture content and other characteristics of base course.
4. The type of subgrade, such as clay, loam and gravel, etc.
5. Moisture content and other characteristics of subgrade.

B. Knowledge concerning the stresses and deformation within a subgrade when acted upon by wheel load pressures under the various conditions listed under A.

Design and Construction of Asphalt Surfaces in the State of Oklahoma

By

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Oklahoma State Highway Commission

In the preparation of this report no attempt has been made to include minute details of construction, and design practices. It is intended, only, to be a brief outline showing the general practices as followed in the design and construction of asphalt surfaces in the State of Oklahoma.

SUBGRADES

The old roadbeds which have been carrying traffic for several years are prepared for the application of a base course and a bituminous surface by improving drainage conditions where necessary. In a few cases sandy roads have been stabilized by the use of asphalt emulsions, MC Cut-back Asphalts, or road oil. These stabilized subgrades then serve as bases for bituminous surface treatments.

In practically all new road construction the subgrades are stabilized by placing the earth in layers, and compacting to maximum density at optimum moisture content. Where a variety of soils is available, the most satisfactory material is selected for use as subgrade material by means of preliminary soil surveys.

BASES

Base courses for bituminous surfaces are, for the most part, constructed of crushed stone or gravel with a soil binder having a Plasticity Index of 4 to 12, with a tendency to hold it nearer the lower limit. The thickness of bases varies from 2 to 4″ in the Western part of Oklahoma, where subgrade and moisture conditions are generally more favorable, to 6 or 7″ in the Eastern part of the State, where subgrade and weather conditions are less favorable. Occasionally Portland Cement Concrete, Bituminous Macadam and Water-bound Macadam are used as bases for bituminous surfaces.

SURFACES

Included in the types of surfacing placed in Oklahoma are Single Bituminous Surface Treatment, Double Bituminous Surface Treatment, using asphalt cement; Road-mixed Mats using SC, MC, and RC materials; Dense graded bituminous surfaces mixed in traveling plants using MC and RC Cut-back Asphalt, and Asphaltic Emulsions; Dense graded bituminous surfaces mixed in central mixing plants for the higher type work, using Asphalt Cements, Asphaltic Emulsions, and RC and MC Cut-backs The trend during the last few years has been to improve the quality and thickness of the base courses, and reduce the thickness of bituminous wearing surfaces.

AGGREGATES

In the construction of bituminous surfaces, local aggregates are generally used. Aggregates from commercial sources are shipped in only when local materials are not adaptable. Sand, screened gravel, crushed gravel and crushed stone, or combinations of these materials are used. Both crushed and uncrushed gravels are permitted, crushed gravel generally being specified when the surface is to be placed on highways which have a high traffic count.

The preparation of the aggregate varies, depending on the type of surface desired. On secondary, and minor highways, the preparation may consist of merely removing the oversize material from pit run gravel. On more important work the aggregate may be screened into as many as 4 or 5 sizes, which are then recombined in the desired proportion prior to mixing with the Bituminous material.

BITUMINOUS MATERIALS

Specifications covering the Bituminous materials correspond closely with those recommended by the Asphalt Institute. SC Road Oil, MC and RC Cut-back Asphalts, Asphalt Emulsions, and Asphaltic Cements are used. The type and grade of Bituminous material to be used is generally dependent upon the aggregate available, and quality of surface desired. SC Road Oils, and MC Cut-back Asphalts are used with aggregates having a relatively high percentage of material passing the No. 200 sieve, while RC Cut-backs, and Asphaltic Emulsions are used in relatively open graded mixtures.

DESIGN OF MIXTURES

For several years the design of the bituminous mixtures has been based on stability tests performed in the Materials Laboratory. Although much progress has been made, and very satisfactory results have been obtained, we are not as yet completely satisfied with the procedure, and are still conducting research to improve our methods of design. Two types of stability testing equipment are being used — the Hubbard-Field Stability Testing Machine, and the Hveem Stabilometer. The chief difficulty encountered in each type of equipment is in attempting to reproduce the compaction and curing obtained by the material under actual service conditions. Our present practice is to cure the molded specimens for a period of 14 days at a temperature of 140° F. prior to testing in the stability machine. Further tests are to be conducted to determine whether or not this method of curing can be improved. In compacting the specimens it was found that unless each specimen was subjected to identical treatment in rodding and compressing, the results could not be correlated. In place of the tamping rod we have been using a compaction hammer, similar to the one used in Proctor Soil Density determinations, and have found it to yield reproducible results. The Hveem stabilometer is not very sensitive to changes in bitumen content of a mixture, but is very sensitive to changes in aggregate characteristics, while the Hubbard-Field Test is very sensitive to changes in bitumen content. We believe that further research will develop a method by which these two pieces of equipment may be used to determine the best possible mixture with any given combination of aggregates and bituminous binders.

MIXING AND PLACING

Mixing is accomplished by the use of motor patrols, traveling plants, or central mixing plants, depending upon the type of surface desired. Spreading is done either by blade or finishing machine. The question of curing time which should elapse between mixing and spreading of the bituminous mixture is of importance. There have been quite a few cases where the mixture was spread and compacted before sufficiently aerated, with the resultant necessity of reprocessing the surface. We believe that field tests to determine the amount of volatile constituents remaining in the mixture will be of value in determining the proper time for placing and compacting the bituminous mixture, and intend to exercise such control on all important work in the future.

Review of Bituminous Highways in Alabama

By

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Alabama State Highway Department

In the State of Alabama, where in 1933 we only had about 100 miles of low cost bituminous highways and have since been building so rapidly, that we now have over 3,000 miles and are still increasing the bituminous surfaced roads at the rate of about 800 miles a year, with the outlook that even this rate will be increased during the next few years, we find ourselves very much interested in the improvements in design that take place from year to year. In fact as we near the completion of work each season we wonder if our type of work has reached its height of perfection, only to find that in starting a new season, we are ready to make some improvements in our design again. As each year goes by we find our organization more and more confident in the work being done, and hence with improvements in construction details as reflected in each new finished project, we find ourselves making improvements in design which seem to come about as a most natural course of events.

The fundamental principles of design of the low cost bituminous surfacing work in this state are as follows:

1. Properly constructed and compacted roadways, made by selecting the best materials economically available for the embankments.
2. Proper handling of all drainage and seepage problems.
3. Placing insulation layers between the graded roadbed and the base course, 6 to 12 inches thick over all sections of questionable subgrades usually of the plastic A-4 to A-7 group of soils. Such material generally is friable, pervious, selected soil.
4. Placing a base course of selected natural graded earth materials in which the percentage of clay, the gradation of the soil fines and the soil characteristics are very, very carefully and closely controlled.
5. Using a surface course that is as nearly water-proof as possible, is flexible and will resist the actions of traffic and weathering.

6. Thorough compaction of each phase of the work, an item that is being recognized as a very important contribution to successful results.
7. Necessity for handling the construction under intelligent engineering control, which must be rigid enough to see that the proper results are obtained, and must be reasonable enough to see that no unnecessary work is asked for which may have a tendency to increase the cost of the work, and more seriously, which may usually cause unsatisfactory work.

There are two important phases of our design which are still being accomplished by judgment of the Engineers, without apparently anything being done to devise a method to assist the Engineer in making his judgments. These two items are:

1. Determining the thickness of the roadbed topping and base course.
2. Deciding whether it is economically justifiable to place base course materials the full width of the graded roadway section from shoulder line to shoulder line, or in lieu of this whether to place the base course material one or two feet wider than the bituminous surface course, and use selected excavation material for the remainder of the distance to the outside edge of the shoulder line.

Our present work consists of grading a road on new location, or regrading an old roadway to a roadway width of 30 feet. While this width is our present minimum, we have recently adopted a new policy of using a 36 foot roadway width, whenever economically justifiable, which is usually everywhere except in very rough country (for our part of these United States), or where right of way complications interfere with such widening or where traffic is light. During the grading operation, of course, all necessary waterway drainage structures are installed. As the grading work progresses, we also install under-drains in roadway cuts or ditches where seepage water occurs, which if not properly drained will cause wet places in the roadway. Such places must be dried up in order to prevent failures of the base course caused by the water softening the base course material to the point where it loses its supporting value.

While grading operations are in progress, tests are constantly being made to determine the class of soil being encountered. Whenever feasible the best soil is placed in the top one or two feet of the roadway section, both in cuts and on fills. Whenever such selection of soil is not economical, the entire roadway is graded with the soil from the adjacent cuts or borrow pits and the roadway is finished below profile subgrade to a depth sufficient to permit the placing of selected friable, pervious material usually of the A-2 group in order to bring the roadway to profile subgrade elevation. This latter operation is called Roadbed Topping. This year we started using the A.A.S.H.O. Compaction Test for controlling the degree of compaction of all work. In embankment work a minimum of 90% of the theoretical compaction is required. This is not difficult to obtain as sometimes under favorable conditions we have obtained as high as 120% actual compaction as compared with the theoretically tested sample.

We are firmly convinced that better compaction of our embankments has added a good deal to the durability of our surface courses. It might be mentioned in this connection, that since we started using the compaction test control we have progressed a long way towards eliminating soft spongy subgrades. The reason for this is, that usually the bed soils (A-7's as a rule) cannot be compacted to 90%, if they contain too much moisture, and therefore by causing such soils to dry some, they can be compacted so that they remain more stable under average conditions.

I wish we were in position to give out a formula for determining the thickness of the total amount of material such as roadbed toppings, base course and surface course, that should be used in order to obtain a satisfactory bituminous surface with each class of subgrade soils encountered. Such wish is far from realization, hence we are trying to do the next best thing. We keep accurate records of the types of soil used on each project, and the kind and thickness of base course used on each project. When a new project is proposed, we try to compare the conditions on this new project with those of projects already finished, and then decide on the thickness of roadbed topping and base course to use. In order to design the safest economical base, this phase is studied by the Division Engineer and his staff, the Materials or Chief Soils Engineer, and his staff, the Bituminous Engineer, and finally by the State Construction Engineer before final decisions are made as to the amount and kind of roadbed topping to be used, and the amount and kind of base course to be used. With regard to kind of material, there is seldom a choice, since we generally have only one class of base course material available in any one locality, and so far we have made the base courses of strictly local materials, except in rare cases. The roadbed topping material is usually the best obtainable within reasonable hauls.

More and more attention is being given to the selection of roadway grading materials, topping materials and base materials. Their proper use in construction is being stressed, such as moisture control, compaction control and uniformity of placing. The latter item is controlled either by proper selection of the material before placing, or by the proper mixing on the roadway with discs, harrows, plows and road machines, or by using both methods. This is particularly true of base course materials. The training of our field men along these lines is one of our present big undertakings. It is very encouraging, however, to note the improvement in quality of work among the Engineers as soon as they become more familiar with the theories or methods of soil selection and proper construction.

For our graded earth natural base course material we now use the following specification:

The Base Course Material shall be well graded between the limits specified and shall conform to the following:

Requirements Total Passing	Percentage by Weight Gravel	Sand-Clay or Top Soil
2-inch sieve	100	
1½-inch sieve	75 to 100	
No. 4 sieve	35 to 85	
No. 10 sieve	30 to 75	65 to 100

The material passing 10-mesh sieve, classed as soil binder, shall conform to the following:

Requirements Total Passing	Percentage by Weight Gravel	Sand-Clay or Top Soil
No. 40 sieve	35 to 95	35 to 85
No. 60 sieve	28 to 75	28 to 65
No. 200 (combined silt and clay)	12 to 39	12 to 35
Clay (8 minutes suspension by elutriation test)	9 to 23	9 to 23

It is expected that the fraction passing the No. 200 sieve (combined silt and clay) will in general not be greater than ½ the fraction passing the No. 40 sieve (soil fines). The fraction passing the No. 40 sieve (soil fines) shall have a liquid limit not greater than 25 and a plasticity index not greater than 6. Any material having a liquid limit greater than 25 and a plasticity index greater than 6, may be used provided the volume change and lineal shrinkage from a moisture content equal to the field moisture equivalent shall not exceed 10 and 3.2 respectively.

We have adopted a policy of using not less than 8 inches of base course material, and in many cases not more than 8 inches either. Where additional thickness is deemed advisable, we add to or thicken the roadbed topping course, and sometimes improve the surface course design. The material used in the base course is very important, and too much stress cannot be given to the basic principle of design, which is the use of the proper proportions of clay, sand and silt in the combined materials. The amount of aggregate apparently is not a governing factor although it adds much to the stability of the finished base. We have concluded from our experience that generally speaking, regardless of the amount of aggregate in the base, the quality of the material below the 10-mesh sieve, which is called soil mortar, must be the same.

In the construction of the base course our engineers now watch the subgrade very closely, so that it is a thing of the past to inspect a project and find the contractor placing base course material on a soft, spongy, loose or extremely dusty subgrade. At present it is our practice to place all base courses in two layers. The first layer is dumped directly on the compacted subgrade which has first been brought to the proper cross-section by machining if on a newly constructed grade, or by the use of levelling courses if placed on an old roadbed. The levelling course materials are usually the same as those used in the base course. Each course is placed, spread, mixed, and rolled before the next course is placed. We do not use form boards for guides. The engineering contro exercised by the use of grade stakes, strings and templets and the advancements in machinery design and operation is such that we can finish the base course to a satisfactory riding grade and cross-section with a motor patrol, or other similar equipment without the necessity of using forms, and at the same time obtain a base course of proper thickness.

All the mixing is done on the roadway by windrowing, plowing and harrowing. Different materials handle differently; sometimes it is necessary to windrow the materials; sometimes it is necessary to wet the material before mixing; or it may require mixing the material while in its natural state of moisture and then wetting the material and mixing again. While all this sounds complicated, the art has been so developed that a contractor can place and mix as much as 3,000 cubic yards of base course material in a day. One of the biggest problems is to shape the base course before priming. At first we just machined the road and then primed same. This was not satisfactory because the road was not always to the proper shape and cross-section. Then we started to scarify the top of all base courses and reshape the loose material to the proper section, and recompact same. This was not entirely satisfactory either because many of the projects were not sufficiently recompacted. Also, during the reshaping there developed a lot of sandy spots that would ravel after the prime was placed. Our present recommended practice is to start shaping, wetting and rolling the base course as soon as it is mixed, and then continue the watering so that the base course will not dry during the compaction period. By the careful use of the templet in connection with the machining operations we can get the base course in proper shape and hold same in that shape during the compaction period so that no additional work, except normal machining, wetting and rolling, is needed before the prime is placed. When the base course material is sandy and contains only the minimum amount of clay binder, one must be careful in machining so as to prevent segregation and loss of the clay binder.

The method of holding the base in shape during the compaction and maintenance period is not always used on top soil and sand-clay base courses. Such bases generally do not have any material above the 10-mesh sieve; hence, particularly while the material is setting-up, we find that after rains, if the machining is not done while the material has the proper moisture content, the material has a tendency to laminate which will produce scabs that will flake off when the base is cleaned for priming. When this condition occurs, we find it necessary to scarify the base about 3 inches, preferably adding a small amount of new base course material, then remix while dry and reshape same. We then use artificial traffic (either trucks or pneumatic-tired rollers) to recompact the base course. After a reasonable amount of such traffic the base is wetted and compaction continued, until properly compacted. The machining, shaping and rolling are carried on simultaneously until the base is ready for priming. This is one of the hardest operations we have and unless the work is done by experienced men it will not be very successful. All the above is mentioned so as to emphasize the fact that after the fundamentals are properly worked out, the work of construction must be developed as an art until construction details are properly cared for.

Our pavement consists of first cleaning the base course and applying a prime coat. This application is applied with distributors at the rate of approximately 3/10 of a gallon per square yard. With improvements in the compaction of the base, we have found that we are able to use less material in this application. In this state we use tar for the prime coat.

In our inverted penetration or surface treatment course, we use two types of treatments. In type one, we use a minimum of .4 gallon hot application covered with ½ cubic foot of 1 inch to ½ inch aggregate. For type two we use a minimum of .35 gallon covered with .45 cubic foot of ¾ inch to ⅜ inch aggregate. All figures are rates per square yard. The hot application is either 150 to 200 penetration asphalt or a heavy grade of tar such as Grade RT 12. The aggregate is either round gravel, crushed slag or stone. The use of the smaller aggregate reduces the cost about 1½ cents per square yard. Also, it is easier to get a satisfactory mat with the smaller aggregate. However, a more stable and durable mat is obtained with the type one treatment.

In the past our wearing course application has been a drag seal made with .42 gallon of cut-back asphalt or tar in two applications with ¼ cubic feet of ⅜ inch to 16-mesh aggregate spread between the applications, then dragged, rolled and sanded. While such seal was fairly satisfactory, we encountered much whipping of the aggregate when the work was done in the fall of the year. Also, we felt like it was not worth the cost—generally 8 cents to 10 cents per square yard, hence this type of seal is not used any more. Our present design is to finish the surface treatment by an application of cold plant mix material. We started three years ago on this work, using 30 pounds per square yard, this year the minimum has been raised to 40 pounds per square yard, and for next year we have set 50 pounds as the minimum. This application fills the larger voids of the surface treatment and provides a more densely graded wearing surface that does not wear tires so much and, furthermore, prevents wear of the surface treatment aggregate. The plant mix seal also retards the weathering of the bituminous materials in the hot application because the finished surface does not have the large surface area to weather as is the case in the very open type surfaces.

The specifications for the plant mix seal is as follows:

Passing ½ inch sieve	100%
Passing ⅜ inch sieve	85 to 100%
Passing No. 4 sieve	40 to 90%
Passing No. 10 sieve	20 to 70%
Passing No. 40 sieve	5 to 35%
Passing No. 200 sieve	0 to 10%
Bitumen and Liquefier	4½ to 8%

We now use mostly 85 to 100 penetration asphalt and a liquefier. Sometimes we use MC-5 or RC-4 and sometimes 120 to 150 penetration asphalt. For all mixtures we now require dried aggregate and if the mixture is to be shipped in railroad cars the aggregate must be cooled. When the mix is made on the project, we have used 150 to 200 penetration asphalt without the liquefier, and do the mixing at about 200° F.

We have not made any changes this year in specification or test requirements for aggregate or asphaltic materials, because we seem to be able to control same with our present specifications, as evidenced by the results we obtain. However, I am sure we would be in position to obtain better results if we made use of the swell test, stripping test, weathering and stability tests. We believe we are getting better plant mix seal mixtures by using the softer asphalts and perhaps by trying to get better graded aggregate. However, as long as we get a 50 pound per square yard plant mix seal placed on the road for about 10 cents per square yard, we hesitate to make any changes that might affect this price structure, especially so when the work we are getting is proving satisfactory. The price mentioned above, includes a tack coat, furnishing the aggregate and asphalt, mixing, placing, rolling and finishing.

We have made several investigations in regard to stabilization, and in our state it still appears that the use of local base course material, properly used and proportioned, is the most economical type of construction. Our present work does not include the addition of any chemical stabilization agents, such as cement, bituminous materials, salts, etc.

We have done some interesting experimental work this year in covering old badly deteriorated and disintegrated concrete pavements. On our heaviest travelled highway (the Montgomery-Birmingham Road) we first used a hammer, attached to a gas operated power shovel, to break the concrete surface into small sections (usually less than one square foot in area). This was done to prevent any further settlement in the pavement, due to shrinkage of the subgrade during the past 10 years, and also to completely break up the pavement so that it cannot act as a slab which might have a further tendency to expand and contract. The next operation was to place a base course of 8 inch minimum thickness when compacted. After the base course was compacted, we placed our type one inverted penetration and covered this with a wearing course of 50 pounds per square yard of plant mix seal material. This section will be under observation until next year and if successful, we will continue this type of reconstruction on additional sections of concrete pavement which are badly disintegrated, or which are breaking up too rapidly.

We have also done some work this year consisting of placing a 40 to 60 pound per square yard plant mix wearing surface over our older drag seal sections, and so far we believe we can get at least five years of good service from such treatment without the necessity of any maintenance except the normal amount necessary for most average surfaces.

Some experimental work has been done using tar, cut-back asphalt, or asphalt cement covered with chips in an effort to produce non-skid surfaces over old slick pavements.

We have also done some work covering over old concrete pavements using penetration surfaces, amiesite type cold mix surfaces, and dense graded cold asphaltic concrete surfaces. All are under observation now.

Principles of Design of Bituminous Highways in Louisiana

By

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In order to present a better picture briefly, of the construction of asphaltic surfaces in this state, we have selected two adjoining projects, the total length of which is twenty-three miles, located in the northern part of the state, somewhat rolling, affording excellent drainage. This type of construction is typical of the greatest mileage of asphaltic construction in this state.

PREPARATION OF ROADWAY

As previously mentioned this section is somewhat rolling and it was necessary to handle a large yardage of dirt. Some of the excavations or cuts are twenty-two feet in depth and the fills as high as twenty-seven feet. Soils used for the embankments were of various types, A-4 and A-7 predominating. Embankments were constructed with four to one side slopes, two to one back slopes, and a three foot ditch with a minimum depth of two feet.

Prior to construction of the embankment, soil samples were taken of the available materials and tests were run in the laboratory to determine the best types. As the density of the fills was to be controlled by compacting at the optimum moisture content of the soil, compaction curves were run on the accepted materials and furnished to the engineer. A minimum compaction of ninety-five per cent was specified for this project. The required compaction of each layer was determined by the "Sand Displacement" method.

All fills were constructed in horizontal layers not exceeding eight inches, loose depth, and rolled with a "Sheeps Foot" roller until maximum compaction obtained. When the moisture content was lower than the optimum, the fills were sprinkled and, if higher, no additional layer was permitted until the material dried out sufficiently or had been mechanically dried by rolling.

On one section of the project, approximately fifteeen thousand feet in length, a fine silt was encountered. This material was thoroughly compacted and a suitable clay over-hauled from hills on each side so that the outside two feet were built with clay and the fine silt trapped within. This prevented erosion.

CONSTRUCTION BASE COURSE

The base course consists of "Local Iron Ore Gravel," eight inches loose, and "Washed Gravel" one and one-half inches loose was added as a stabilizer. After cutting the subgrade to line and grade, iron-ore was placed in two layers; the first portion was bladed, wet and rolled with a ten-ton, three-wheeled roller. Before placing the second layer, the compacted iron-ore was scarified lightly to insure bonding and the last layer was deposited, rolled and bladed to the desired cross section, the washed gravel stabilizer was then placed and rolled into the base. The finished base was sprinkled, rolled and opened to traffic for thirty days and, at the end of this period, all loose material was rolled into the base in preparation for the application of asphaltic surfacing.

APPLYING SURFACE COURSE

An asphalt prime was added to a swept base at the rate of three-tenths gallon per square yard of surface and, after twenty-four hours, the road was opened to traffic for five days. At this time penetration of the prime averaged one-fourth of an inch.

Application of the surface treatment was as follows:

An asphalt cement, two hundred—two hundred and fifty penetration, at a temperature of two hundred and fifty to three hundred and twenty-five degrees, was applied to a firm base at the rate of four-tenths gallon per square yard of surface, and immediately covered with coarse gravel measuring one cubic yard for each fifty square yards of surface. This course was then hand-broomed, drag-broomed and rolled with a five-ton, three wheeled roller.

The second application of hot asphalt was applied at the rate of three-tenths gallon per square yard and covered with an intermediate size gravel measuring one cubic yard per hundred square yards of surface, and broomed and rolled as in the preceding course.

The third application of asphalt was applied at two-tenths gallon per square yard, then covered with a seal course gravel measuring one cubic yard per hundred and thirty-five square yards of surface, then broomed and rolled. This surface will average one and one-eighth inch in thickness, two-tenths to twenty-five hundredths gallon prime, nine-tenths to one gallon asphalt cement, and one hundred pounds mineral aggregate per square yard.

The foregoing method of construction has proven very satisfactory.

The majority of the base courses consists of local sand clay gravel meeting the requirements as shown following these remarks. None of the base course supporting surface treatment has any admix, chemical or bituminous.

We have built approximately fifteen miles of sand asphalt using local sand and RC cut-back asphalts. This will undoubtedly be discussed at length by some other member.

Base Course Material for Asphalt Surface Treatment

Screen Size	Per Cent Passing
1½ inch	95 - 100
No. 4	35 - 50
No. 10	25 - 45
No. 40	15 - 35
No. 200	7 - 17
Liquid Limit—Maximum	25
Plastic Index	0 - 6

Bituminous Material for Prime Application (Type P-2): This material shall be free from water and shall meet the following requirements when tested in accordance with the methods hereinafter enumerated:

Viscosity Sec. Saybolt Furol 60 cc. ,at 122° F.	100 - 150
Asphalt content, per cent	55 - 65
Flash Point C.O.C., not less than	175° F.
Distillation by Volume:	
Total distillate to 437° F., Not more than	5
Total distillate to 600° F., Not less than	18
Total distillate to 680° F., Not more than	35
Tests on Residue from Distillation:	
Float test at 122° F., Not less then	150
Per cent soluble in Carbon Tetrachloride Not less than	99.0

Asphalt for Hot Applications: The asphalt shall be homogeneous, free from water, and shall not foam when heated to 175° C. (347° F.) It shall meet the following requirements:

1. Specific gravity 77°/77° F., not less than	1.020
2. Flash Point, not less than	347° F.
3. Penetration at 77° F., 100 grams, 5 seconds	200 to 250
4. Loss at 325° F., 50 grams, 5 hours, not more than	2%
(a) Penetration of residue at 77° F., 100 grams, 5 seconds as compared to penetration before heating, not less than	60%
5. Bitumen (soluble in carbon disulphide), not less than	99.5%
(a) Organic matter insoluble, not more than	0.2%

Mineral Aggregate for Asphalt Surface Treatment

Screen size, inch	Coarse Aggregate	Intermediate Size Aggregate	Seal Course Aggregate
Passing 1½ inch	100	100	100
Passing 1 inch	90 - 100	100	100
Passing ¾ inch	40 - 70	100	100
Passing ⅝ inch		100	100
Passing ½ inch	Max. 10		100
Passing No. 4		Max. 7	40 - 70
Passing No. 10			0 - 20
Passing No. 50			Max. 5

Bituminous Construction in Kentucky

By

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Kentucky has numerous aggregates available that are suitable for bituminous highway construction. These aggregates vary widely over the state. In Eastern Kentucky there is a predominance of good sandstone and also blast furnace slag. There is an abundance of limestone in Central Kentucky, while in Western Kentucky there are large quantities of bank gravel and river gravel. Considerable quantities of usable creek gravel can be had throughout the state. Hence natural occurrence or local availability is most often the controlling factor in the selection of aggregates, especially for bases. Of course all aggregates must meet certain requirements of grading, soundness, abrasion, and freedom from deleterious substances. Abrasion requirements for base aggregates are five per cent lower than for surface aggregates. Otherwise the specifications are the same for both.

For fine aggregates Kentucky has river sand, bank or pit sand, and limited quantities of fine grain quartz conglomerate deposited in ledges on various mountains. River sand is the fine aggregate most often used.

Kentucky has about sixty miles of bituminous stabilized bases. These bases consist of treating the soil and aggregate as it exists on traffic bound roads without the addition of other granular material. Other than these, Kentucky has a limited mileage of bituminous bases.

Water-bound macadam or traffic-bound base courses are most widely used. These bases are covered with from one to three inches of binder course, consisting of an open texture bituminous mix, and one-half to two inches of surface course of some type.

Surfaces used are coarse, medium, and fine textured. Dense graded surfaces are used in and near most cities. Natural rock asphalt is widely used for surfacing.

Several types of surface construction are used including hot mixed-hot laid, hot mixed-cold laid, cold mixed-cold laid, and mixed-in-place surfaces.

Crushed limestone has been generally used for surface aggregate. Blast furnace slag has given excellent results when used in surfaces. Uncrushed gravel and sand have been successfully used for medium textured hot mixed-hot laid surfaces. Several mixed-in-place surfaces constructed with uncrushed gravel have not been satisfactory. However, later experimental projects have indicated several methods for using uncrushed bank and creek gravel with medium curing cutbacks. By using 20% to 30% sand in the mix to get a more uniformly graded aggregate from 1¼″ down, better surfaces were obtained. Two course construction offers another alternative. The bottom course is mixed in place using MC-3 and uncrushed gravel. This is covered with a ½″ surface treatment using fine limestone chips. Perhaps the simplest satisfactory procedure has been to seal the gravel surface with .2 gallon of MC-3 and 15 pounds of sand per square yard.

Selection of the type base and surface to be constructed on a particular project is dependent on the classification of the road as determined by the Planning Survey.

The thickness of the base is determined by assumed concentrated loads distributed on 45 degree lines to the subgrade having an assumed bearing value, all based on the best obtainable data. This applies principally to high type construction and is used as a guide on secondary roads, together with actual knowledge by experience.

Recently, in base design, consideration is being given to the type of soil in the subgrade. Subgrade soil surveys and profiles are new in this state and therefore no definite reqirements have been established between soil types and base thicknesses.

Climatic variation is one of the chief factors in the selection of the grade or type of bituminous material to be used. With surface treatments using cutback materials, special emphasis is placed on the season of the year in the selection of the proper grade.

In recent specifications the change has been made from Deval Abrasion to Los Angles as standard. This was done because preliminary tests indicated a better correlation between service records and the Los Angeles. The Los Angeles also had the advantage of permitting tests on the final crushed product as used, as well as on ledge rock. Kentucky uses gradings A and B and standard procedure as recommended by A.A.S.H.O. for most crushed materials. On materials finer than ⅜″ round, the grading and procedure developed by the Texas Highway Department is used. However, this state still samples and tests all ledges before they are approved for crushing.

Five cycles on graded material in Na_2SO_4 are now used for soundness in preference to previous five cycles on one size aggregate. This permits a more representative sample and gives more reproducible results. In addition to using 1½″, ¾″, ⅜″, No. 4 screen as recommended by A.S.T.M. for determining the loss, 1″, ½″, No. 4, No. 8 are also being used. It was found that many particles that fractured in only one plane and did not actually disintegrate would pass the larger screens. This often gives large losses on samples that have satisfactory service records. It is felt that use of the smaller screens will eliminate this. Sufficient data will soon be available to correlate percentages of loss with service records and then the smaller screens will be incorporated in the specifications.

Specifications have for some time permitted the use of soft penetration grade oil asphalts but only recently has there been a trend away from 40-60 toward 85-120 grades. This is being done trying to obtain more durable and flexible surfaces, especially in sections subject to large temperature changes. The softer grades are also being used over flexible bases.

Much interest is shown and some work has been done in the department determining constants on bituminous materials at low temperatures. As yet sufficient data has not been accumulated to warrant adoption of any specifications of this type.

Few changes have been made recently in plant mixes and their control. There has been a definite lowering of aggregate temperatures and mixing temperatures. The mixing time has also been shortened. These changes were made endeavoring to decrease the asphalt hardening taking place before the mix is laid.

It should be mentioned that results using petroleum naphtha liquefiers in cold mixed-cold laid bituminous concrete have been very unsatisfactory and their use is being gradually discontinued in favor of heavy medium curing cutbacks.

On surface treatment and resurfacing work, much closer control has been exercised during the past few seasons. Each month all engineers doing bituminous work are furnished with a summarized sheet on cutbacks. (See specimen). This sheet gives the engineers data such as specific gravity, weight per gallon, and base asphalt content which enables them to more easily and accurately figure quantities of bituminous material. For the mixtures ordinarily used the quantities are put on this sheet. This sheet is issued to help obtain more uniform surfaces from job to job and obtain uniform results using cutbacks furnished by different producers using varying volumes of distillate. This control helps prevent the use of excess bituminous material. Excess asphalt has previously caused much trouble by bleeding and resultant slick surfaces throughout the state. From 3.5% to 4.5% asphalt is now being used in surface treatments. All bituminous contents are expressed as percentage of base asphalt in the total mix. So far no troubles have developed from lean surfaces.

This department is starting the use of soil surveys, soil profiles having been run on only a limited number of projects. A soil map of the whole state is now being compiled by the soil department, this map to show the sections where various types of soil occur and reoccur. It is hoped that by the use of this map much duplication can be avoided in soil surveys; and experience gained with a particular soil at one place can be used more readily when this soil is found at another location.

This department now has in the specifications provisions for extra compaction on grade-drain and embankment projects. This is effected only when so stated on the plans for a particular project. It requires that suitable soils be used and compacted to 90% of theoretical maximum density as determined by Proctor methods. Consideration is being given to adopting compaction specifications about as outlined by A.A.S.H.O. and B.P.R., requiring varying percentages of theoretical compaction depending on the type of soil and weight per cu. ft. of the soil.

During the past few seasons this department has constructed several experimental projects trying to obtain pertinent information that will permit the construction of cheaper and better surfaces in the future by using materials locally available and also by using some slightly different methods of construction. The details of some of these projects should be of general interest.

In June 1938, a project was constructed to determine whether or not large quantities of waste screenings around limestone quarries could be successfully utilized for bituminous surface treatments. This project was divided into four sections.

Section No. 1 — Length 300 Feet.

The gradation of the aggregate used was as follows:

	Passing	Retained
Sieve No. 4	88.2%	11.8%
Sieve No. 10	41.4	46.8
Sieve No. 40	14.4	27.0
Sieve No. 80	10.3	4.1
Sieve No. 200	6.7	3.6

The treatment was 60 pounds of aggregate per square yard and .65 gallon of MC-3 per sq. yd. Extraction of the final mixture showed a bituminous content of 4.6%. Samples of the aggregate contained 1.26% moisture when the bituminous material was applied.

The road was first primed and then stone spread on the road from truck tail gates. A multiple blade maintainer was run through the stone three complete trips to eliminate as much excess moisture as possible. The stone was then shot with .4 gallon per square yard of MC-3. The maintainer made one trip, followed by a grader which cut the material to each quarter section. The grader then cut these quarter sections to the center of the road. It was then necessary to spot prime small sections of fine dust in surface irregularities that the maintainer and blade could not reach. The maintainer was again run through the windrow in the center of the road and grader followed, cutting the mix to the quarter section. Again the grader threw the windrow to the center. This procedure was repeated until mixing was complete. The mix was then laid to section and rolled. No seal was applied. This made a very dense, non-skid surface.

Section No. 2 — Length 600 Feet.

This section was constructed from waste of almost the same gradations as No. 1. However the mix was rained on before being laid out and additional bituminous material was added the following day. The total application was not recorded. This section bled so badly that it was heavily chipped and rolled shortly after being laid. This section was therefore of little experimental value.

Section No. 3 — Length 208 Feet.

The treatment and aggregate used were the same as in Section No. 1. The same construction procedure was followed. The bituminous content was increased to .76 gallon of MC-3 for 60 pounds of quarry waste. The mix showed 8% asphalt by extraction. This section was much too rich with bituminous material and after completion was chipped and rolled.

Section No. 4 — Length 50 Feet.

This section was not primed, since considerable material had been carried onto it from the previous rich sections. Quarry waste and MC-3 were also used on this section. Here they were mixed in a small concrete batch mixer. Five hundred pounds of waste was used per batch. Sufficient quantity of MC-3 was added to coat all the aggregate. The final mix had 4.2% asphalt. The mixing time was three minutes per batch. The aggregate contained 1.14% moisture when mixed. The mixed material was windrowed in the center of the road, spread and rolled with a ten-ton roller.

In the summer of 1939, all sections constructed with quarry waste are in good condition and not one has developed a slick surface. So far this project indicates that cheap quarry waste can be satisfactorily used for surface treatment.

Because of the success of the short section constructed with the concrete mixer as described above and one or two other short trial sections, considerable mileage is being constructed this season using a similar but larger scale set-up. All of the equipment used is readily portable. A Rex 27-E concrete mixer is used in conjunction with an Adnun Bituminous Paver. The concrete mixer is set up stationary at the stock pile. The mix is proportioned by volume. The aggregate is loaded into the mixer skip with a ⅜ cu. yd. shovel mounted on a F.W.D. truck. The bituminous material is trucked to the job and transferred to a supply tank parked near the mixer. From the supply tank it is pumped to a tank on top of the mixer where the predetermined quantity is added to each batch. The bituminous material is passed into the mixing drum through the same opening as the aggregate and simultaneously with the aggregate. Four thousand pound batches are mixed. Seventy-five seconds is generally sufficient mixing time. The finished mix is trucked to the paver. It should be noted that further mixing is effected by the paver during the laying operations. The base asphalt content has been varied between 4% and 6%.

Two projects have been constructed so far. Sixty pounds of aggregate per square yard were applied. Aggregate used on both projects met the following gradation requirements.

Passing ½″ square	100%
Passing ⅜″ square	80 - 100%
Passing No. 4	10 - 30%
Passing No. 8	0 - 5%
Passing No. 20	0 - 3%

The first project was approximately 10 miles in length. The bituminous material was MC-3. This road was heavily traveled and was kept open to traffic. Trouble was encountered because the MC-3 did not set fast enough to permit immediate rolling behind the paver and also the mix "picked-up" badly under traffic even after being rolled. Considerable shoving of the mix was also encountered on one steep hill. Covering the newly completed surface with sand and rolling helped stop the "picking-up."

The second project was 15 miles in length. RC-4 was selected as the bituminous material to be used. This mix could be rolled immediately behind the paver. No trouble was caused by traffic. Coating trouble appeared when damp stone was used. It was decided that a less viscous material might coat the damp aggregate more thoroughly. Consequently RC-4 was discontinued in favor of RC-3. This worked very nicely but still did not satisfactorily coat wet aggregate. If the surface laid with wet aggregate is not satisfactory, the problem may be solved by devising some method of precoating with a small amount of some material such as MC-1. Where it is not necessary to keep the road open to traffic, the problem can be solved by using emulsion.

All the second project was sealed during construction by hand spreading and hand brooming 15 pounds of sand per square yard over the surface. Several methods of applying the sand were tried. The most satisfactory was to spread the sand immediately behind the paver before the mix was rolled the first time.

Attempts were made to use sand in the mixture along with the stone. Several difficulties presented themselves. Segregation took place. The fine material balled. The sand was preferentially coated. Part of the mixture would remain in the mixing drum. This practice therefore was discontinued.

Cost analyses are not complete on these projects but estimates indicate that the cost will compare favorably with similar road mix surface treatment.

The projects constructed this way have resulted in very good riding surfaces. However it is believed that a 75 pound treatment should be the minimum in order to prevent thin places on top of irregularities in the old surface.

Of course this construction is still very new but is very encouraging. Plans are to use this equipment to place a cover course on about twenty-five miles of old concrete pavement during this season.

In 1937 a 6 mile blade mix surface treatment experimental project was constructed to compare various bituminous materials, to compare varying quantities of each material, and to try the use of quarry waste and river sand for fines in the mix. Medium tar, MC-3, and RC-2 were used.

The project was divided into twenty-four ¼ mile sections. The total treatment on all sections was 60 pounds per square yard. Two sizes of stone were used, namely from ½″ square to No. 8 sieve and from ⅜″ square to No. 20 sieve. Class A concrete sand was used. The quarry waste was graded from ⅜″ square down to 10% passing No. 200 sieve.

Various combinations of aggregates were used, all sections where fines were used being 40 pounds stone and 20 pounds fines per square yard.

The bituminous content was varied from 3.0% to 8.8%. All sections above 5% were too rich and bled freely. All rich sections except those where fines were used developed very slick surfaces. This project definitely discouraged the use of excess quantities of bituminous material. It also indicated that when fines are used, MC-3 is a desirable material. The sections where fines were used do not show as much raveling or other types of wear as do the other sections.

Numerous other projects have since been constructed using sand. One project was constructed in May, 1939, using a very angular quartzite sand uniformly graded from No. 4 to No. 200 sieves. Two sections constructed were as follows:

	Section 1	Section 2
Gals. RC-2 sq. yd.	.667	.530
Pounds No. 9 Stone sq. yd.	45	60
Pounds Sand sq. yd.	30	15

Considerable difficulty was expected in coating this sand by blade mixing due to the large amount of fines and also the extremely smooth surfaces of the aggregate. As was expected, the mixing difficulty was a reality but mixing was finally accomplished. It is believed that the mixing could have been more efficiently accomplished by using a multiple blade maintainer.

It was feared that No. 1 section might be too rich, but in August, 1939, this was the best section, No. 2 showing slight raveling along the edges.

Results from all projects using sand can be summarized as follows:

Both coarse and fine aggregates have been sanded. It has been found that with coarse aggregates 25% and upward of sand can be used; with fine aggregate, 25% is the maximum.

The best construction method used so far on sanded surfaces is as follows:

The aggregate is shot with the proper amount of bituminous material and thoroughly mixed and laid out. The desired quantity of sand is then spread over this mix. If the sand is wet when applied, it is allowed to dry before being mixed with the coarse aggregate. After the sand is dry, it is incorporated into the coarse aggregate by whatever method of mixing is being used. The sand will coat without the addition of more bituminous material. Experience has shown that practically no segregation takes place. After the mixture is all thoroughly coated with bituminous material it is either put in a windrow or laid out and rolled depending on the type cutback used.

The conclusions, based on experience so far, recommend the use of sand or limestone grit as a filler in road mixed surface treatments. The addition of these materials result in more durable, non-skid surfaces than were previously obtained.

Numerous experimental projects have been constructed in Western Kentucky trying to select the most suitable bituminous materials and best construction methods to be used with local uncrushed gravel.

In 1937 a 10-mile project was completed using emulsion. The gravel had an average gradation of:

100% passing 1¼″ round screen.
52% passing ¼″ round screen.

The treatment was 125 pounds per square yard and 1.3 gallon emulsion per square yard. It was also necessary to add about 3 gallons of water per square yard.

The emulsion was applied on top of the windrow and mixed with a traveling triple pug-mill mixer.

Many difficulties were encountered on construction and progress was very slow and very costly.

The use of emulsion for prime was not satisfactory and was discontinued because it did not penetrate.

This project was not successful, as raveling and excess wearing took place soon after its completion. It was covered with an asphalt cutback-limestone aggregate wearing course a few months after completion.

Several other uncrushed gravel projects using various combinations of sand, medium curing cutback, and tar have all indicated that sand must either be incorporated in mix or used as a seal except with two course construction. With two course construction, crushed gravel or crushed limestone must be used for the top course.

Four years ago a four mile project using powdered asphalt and SC-3 flux was constructed for observation purposes. This project is still in excellent condition while in the meantime the road adjoining has had one cutback surface treatment. It was decided to surface this entire road this season. After examination of this experimental project the condition was seen to be so good that the decision was to leave it and surface the rest of the road.

Several units of road mix are now under construction using SC-3 flux and powdered asphalt in such quantities as to obtain a 120 penetration asphalt.

Most all bituminous materials used in Kentucky are positive spot materials. Specifications do not require the use of the spot test. Most of our bituminous materials do have a required solubility of 99 plus or 99.5 plus in CCl_4.

Some work has been done recovering bituminous materials from surface treatments that have been under service varying lengths of time. Abson's Method of recovery was used in making these studies. These tests show that the rapid curing cutbacks used in this state harden rapidly and the ductility decreases to very low values under service. This is in agreement with service behavior. Medium curing materials from one source only showed marked decrease in penetration or ductility. (See accompanying tabulation for comparison of materials.)

This department, like many others, has made an effort to perfect some method of quantitatively measuring the film adhesion of bituminous materials to various aggregates. A modified washing displacement test developed in the laboratory is used. The bituminous coated aggregate is severely washed in water at a definite temperature around 140° F. to 160° F. Most of the displaced bituminous material is removed from the sample proper by the centrifugal force developed during the washing. The sample is not agitated or subjected to abrasion during the test. However, the container for the sample is placed in a number of positions during the washing so that all pieces of aggregate are subjected to the same conditions. The test requires ½ hour after the sample has been coated and cured. The amount of film displacement is measured by the weight of bituminous material lost from the sample. The loss expressed as percentage by weight of asphalt originally present has ranged from 15% to 70%, using the same asphalt and different aggregates. The comparative ratings by this method agree very closely with field observations. This work will be continued.

Kentucky's 60 miles of bituminous base stabilization consists of sections using emulsion, medium tar, MC-3 and SC-3. According to a recent condition survey, failures have developed on some of these projects. (See accompanying summary of condition survey.) Probably the major cause of failures was compacting and sealing the bases with much too high a moisture content. This is substantiated by sections that have been scarified, aerated, recompacted dry without the addition of extra materials, and are so far behaving satisfactorily. Most failures on these bases have occurred during the hot seasons and on first observation appeared to be due to excess bituminous material. In some instances an excess has probably been present but the condition was exaggerated by absorbed moisture becoming free water at summer temperatures. Samples from the richest and worst sections have sufficient Hubbard-Field stability after being dried, and recompacted. It is believed that all of the bases can be reclaimed, with the possible loss of part of the surfaces, by aeration and recompaction. In some places the soil contained too much minus 200 material with a high plastic index to be adaptable to this work and these sections will probably have to be diluted with granular material.

No definite conclusions can be drawn to point out the most satisfactory bituminous materials, but some results indicate that slow curing oils are not desirable and that emulsions are in general better for use with materials having a high percentage of minus 40 or minus 200 with a plastic index about five.

The projects indicate that slow curing oils and medium curing cutbacks do not cure sufficiently when soil fines are present, unless they are placed in the early summer and exposed to considerable hot weather before being sealed.

Types of construction used on these stabilized bases are about as follows:

1. 10 miles applying emulsion through suboiler and compacting without further mixing.

2. 10 miles applying emulsion through suboiler, and mixing with disc harrows and blades before compacting.
3. 11 miles applying slow curing oils (SC-3) with suboiler and thoroughly mixing with disc harrows, disc plows, and blades before compacting.
4. 1 mile using emulsion through Barber Greene traveling plant.
5. 1 mile mixing MC-3 in a Barber Greene traveling plant.
6. 1 mile mixing meduim tar in a Barber Greene traveling plant.
7. 1 mile mixing SC-3 in a Barber Greene traveling plant.
8. 10 miles using SC-3 and a Wood's traveling plant.

None of these methods of construction have been exactly ideal from the standpoint of economics and results. Emulsion applied through the suboiler did not properly disperse unless mixing followed the application. If this is necessary the purpose of the suboiler is at least partially defeated. Application of slow curing oils through the suboiler to be road mixed does not appear to justify the cost involved. Barber Greene traveling plants do a thorough job of mixing but require the use of an undesirable quantity of water to obtain a consistency which enables the machine to handle a large volume of material readily. Woods traveling plants also require an undesirable quantity of water with most soils, and in addition are unable to handle a large volume of material at one pass. However, experience here has shown that they do a thorough job of mixing.

Considering both cost and results obtained so far, it seems that application of bituminous material with a distributor followed by thorough road mixing is probably the most satisfactory and economical method of constructing bituminous stabilized bases in this state, especially where considerable quantities of plastic soil fines are used. The traveling plants work splendidly in sands or very sandy soils.

However it should be noted that in 1937 one 2,000-foot section containing A-6 soil binder was successfully mixed with a Barber Greene without any water except that contained in the scarified soil. This moisture content varied from 8% to 9%. The bituminous material used was SC-3 and the quantity was increased to about 4.8%. If an attempt was made to add even 1% free water to the mixture the machine would not function. If any water at all was added, it was necessary to reach a content of 25% or 30% before obtaining a workable consistency. This high moisture content is objectionable for several reasons some of which are: Time required for the mix to drain and become workable, extra time and manipulation required to compact the mixture, tendency of the water to reduce adhesion, tendency for the excess water to carry away some of the bituminous material, cost of the water, and numerous others. It must be stated that the section mixed in the Barber Greene dry or without the use of additional water is still in excellent condition, though the dispersion was not as complete as when water was added. This mixture was compacted at a moisture content considerably below optimum.

There is a definite trend in surfaces used by this state away from open graded materials toward medium and fine textured surfaces. To obtain these surfaces, various amounts of sand are being incorporated in surface treatments along with finer, more evenly graded aggregates. These are producing surfaces more resistant to fracture and crushing and at the same time having very desirable non-skid properties.

Because of research work done by the laboratory and systematic field observations, this department is rapidly discontinuing the use of RC-2 and using more MC-3 to avoid surface treatments that harden rapidly, becoming brittle and without "life."

To obtain more accurate control, to permit use of more fines, to eliminate stripping by using oil asphalts, to eliminate bleeding, eliminate frequent retreatments, and other troubles encountered with road mixes and surface treatments using cutbacks, Kentucky is rapidly placing more plant mixed surfaces. Dense graded bituminous concrete is placed on heavily traveled roads and medium textured mats on roads with lower traffic counts.

As has before been mentioned in this paper, Kentucky has recently started extensive use of sand with cutbacks for light seals on surface treatments. The results have so far been indeed gratifying, giving excellent riding, non-skid surfaces.

Fine limestone chips are also used with cutbacks for sealing. Blast furnace slag screenings because of their hardness and angularity have been considered for sealing, and have worked nicely in the limited quantities that have been used.

Light treatments of natural rock asphalt are widely used for dense seals in this state.

INFORMATION ISSUED EACH MONTH BY THE LABORATORY ON BITUMINOUS MATERIAL

Producer	Type of Mat.	Spec. Grav.	Weight per gal.	% Base Asphalt by Weight	Gals per 100-lbs. of agg. to obtain 3½ % Base Asph.	Gals. per 100 lbs. of agg. to obtain 4 % Base Asph	Gals. per 100 lbs. of agg. to obtain 4½ % Base Asph.
B	RC-2	1.003	8.34	82.6	0.522	0.606	0.688
B	MC-3	1.021	8.51	91.8	0.461	0.535	0.608
C	RC-2	.987	8.23	82.3	0.531	0.617	0.701
C	MC-3	1.001	8.34	86.8	0.497	0.577	0.656
E	RC-2	.945	7.88	77.3	0.591	0.686	0.780
E	MC-3	.963	8.02	83.7	0.536	0.623	0.708
D	RC-2	9.83	8.18	81.0	0.544	0.630	0.710
D	MC-3	1.000	8.34	86.7	0.498	0.578	0.656
A	MC-3	.953	7.94	83.3	0.544	0.632	0.716
F	RC-2	.958					
F	MC-3	.978	8.15	83.2	0.531	0.616	0.694
G	RC-2	.942	7.85	78.0	0.588	0.683	0.768

TABULATION OF CONSTANTS ON RECOVERED CUTBACK BITUMINOUS MATERIALS 1-39 TO 4-39

ORIGINAL CUTBACK									RECOVERED MATERIAL						
			Distillation												
Producer	Time Under Service	Type Bit. Mat.	225° C.	315° C.	360° C.	Pen. of Res. 25° C.	Duct. of Res. 25° C.	% Sol. in CCl4	No. Samples	Pen. 25° C.	Duct. 25° C.	Soft Pt. ° C. R. & B.	% Sol. CCl4	% Loss 5 hr. 163° C.	Pen. 25° C. After Loss
A	7 Mo.	RC2	16.8	25.2	27.5	94	100+	99.70	2	47	54	58	99.51	0.37	36
B	6 Mo.	RC2	10.4	20.4	22.5	76	60+	99.65	4	38	13	56	99.80	0.51	24
C	7 Mo.	RC2	13.2	20.4	22.7	89	100+	99.62	5	49	69	55	99.79	0.88	29
D	9 Mo.	RC2	15.3	24.3	27.5	84	100+	99.78	4	57	100+	53	99.90	0.66	28
E	11 Mo.	RC2	15.5	25.5	28.5	89	100+	99.85	8	55	83	53	99.77	0.80	40
A	9 Mo.	MC3	0.0	17.5	23.0	190	100+	99.75	2	39	24	61	99.83	0.63	34
C	6 Mo.	MC3	0.0	14.0	20.1	150	100+	99.74	5	103	92	44	99.76	1.90	60
D	8 Mo.	MC3	0.0	10.3	17.0	151	100+	99.63	4	148	100+	45	99.85	0.85	79
E	4 Mo.	MC3	0.0	9.3	16.0	135	100+	99.70	2	42	18	55	99.81	1.14	34
F	6 Mo.	MC3	0.0	12.7	18.0	169	100+	99.65	7	126	100+	45	99.65	1.38	80

All materials recovered by Abson's Method.

RESULTS OF CONDITION SURVEY CONDUCTED MAY, 1939 ON STABILIZED BASES IN KENTUCKY

	Greenup Co. SP 106-AS-a	Greenup Co. SP 106-AS-b	Greenup Co. SP 106-AS-c	Greenup Co. SP 106-AS-d	Greenup Co. SP 106-AS-e	Greenup and Lewis Cos. SP 106-AS-f	Muhlenberg Co. SP 77-AD	McLean Co. SP 2½-DS2	McLean Co. SP 125-AS2	McLean Co. SP 2½-CS2	Martin Co. SP 34-ES
Date Constructed	9-37 11-37	9-37	8-37	8-37 9-37	9-37	9-37 11-37	11-37 1-38	7-38 10-38	6-38 10-38	6-38 10-38	6-38 10-38
Length, miles	9.82	0.18	1.05	0.93	1.04	7.80	7.66	9.78	3.25	5.76	10.59
Type	Sub-oiled	Sub-oiled	Plant Mixed	Plant Mixed	Plant Mixed	Plant Mixed	Plant Mixed	Plant Mixed	Sub-oiled	Sub-oiled	Road Mix
Admixture	Emulsion	SC-3	Emulsion	MC-3	Med. Tar	SC-3	SC-3	SC-3	Emulsion	Emulsion	SC-3
% Satisfactory base and satisfactory surface	2.3		3.8		90.7	20.5	80.7	63.8	99.8	98.0	45.7
% Satisfactory base and slight surface failure							1.6	12.9		0.7	4.9
% Satisfactory base and moderate surface failure							1.6	0.7			11.6
% Satisfactory base and extreme surface failure								0.8	0.2		13.3
Total Satisfactory base, per cent	2.3		3.8		90.7	20.5	83.9	78.1	100.0	98.7	75.5
% Slight base failure and surface failure	9.1		6.9	0.6	7.6	10.0	11.7	14.3		1.3	11.0
% Moderate base failure and surface failure	13.5		27.7	7.8	1.7	29.1	3.5	6.7			4.7
% Extreme base failure and surface failure	75.1	100.0	61.6	91.6		40.4	0.9	0.8			8.8
Average % base moisture	10.5	10.3	10.0	10.1	7.4	10.0	7.6	9.2	11.5	8.9	8.2
Average % subgrade moisture	14.5	13.3	13.3	11.0	8.4	11.8	15.9	14.4	18.4	16.7	11.2
Average difference between base and subgrade	5.2	3.1	3.3	0.9	1.0	1.8	8.4	5.3	6.9	7.8	3.2

1. All moisture contents were run on minus No. 4 material.
2. The section constructed using medium tar was through almost pure sand.

BITUMINOUS ROADS IN WEST VIRGINIA

By

E. N. BLACKWOOD
Construction Engineer
West Virginia State Road Commission

The mileages of bituminous work contracted for by the State Road Commission in Virginia since January 1, 1939, are as follows:

28.0 miles of bituminous concrete.
160 miles of bituminous road mix.
31 miles of bituminous stabilized gravel base course.
171 miles of surface treatment.

BITUMINOUS CONCRETE

Prior to 1937, bituminous concrete was placed in two courses on prepared bases. The first operation was the application of a tar coat of 0.1 to 0.2 gallon per square yard. The second operation was the placing of 1½″ bottom course with ½″ top course, using a finishing machine operating on metal side forms supported by blocks and shims which were not substantial and did not produce uniformly smooth surfaces. In lieu of metal side forms, the contractor was permitted to use strips made of surfacing material built to string-line grade, which did not prove satisfactory due to the delay caused by hand-building of such strips and the expense of same.

The present method, as contrasted with the above procedure, is:

1. Irregularities in the base surface are removed by patching with mixed-in-place material in courses not exceeding 1″ in thickness, each course for a patch being thoroughly compacted.
2. Place 75 to 100 pounds of open graded road mix using a sled runner type drag. The material is then compacted, checked for trueness of surface and repatched until the trueness of surface meets specification requirements for the finished wearing surface.
3. On this carefully prepared bottom course is then placed a bituminous wearing surface ½″ to ¾″ in thickness using a paving machine which, without forms, rides the carefully prepared bottom course.

The resulting surface by the latter method produces a smoother riding surface than could be obtained by the old method of side forms or built up strip and there is a saving in this method,as evidenced by bids received, of $1.00 to $4.00 per ton on the bottom course.

MACHINES

The major improvement of finishing machines is the addition of a tamping device operating immediately behind the strike-off template.

ROAD MIX

Bituminous mixed-in-place surfaces have been placed by the same eneral method and equipment during the past three years. However, hanges in aggregates and bituminous content have been made to correct he deficiencies observed.

Prior to 1936, a dense graded aggregate was used for mixes with ow bituminous content to prevent fat areas caused by segregation of he fines, but the fat areas were not entirely eliminated.

The present method is open graded aggregates, of which the maximum ize is the approximate thickness of the finished course. Bitumen is added, horoughly mixed and is leveled with a sled runner type finishing mahine or simple drag. The voids are then filled with fine chips or sand n two applications of bitumen. The bituminous material has been increased rom 0.8 to 1.0 gallon for each 100 pounds of aggregate including the iller. No fat areas have occurred due to the increase in bitumen, except where road mix voids have been completely filled by traffic or other causes previous to the final application of bitumen.

SURFACE TREATMENTS

No changes in equipment or methods for light surface treatments have been made. The surface treatments of 60 pounds to 75 pounds of aggregate per square yard are built by identical methods and equipment as used for the retread or mixed-in-place. The maximum size of the aggregates used are approximately the same as the thickness of the finished course with a small per cent of material passing the smaller screen. The filling material is identical with that used for the mixed-in-place surfaces; however, it varies from one application of filler and bituminous material on the 60 pounds surface to one application of bituminous material and two applications of filler for the 90 pounds surface, and on the heavy mixed-in-place surfaces there are two applications of bituminous material with three applications of filler.

LIGHT PENETRATION STABILIZING COURSE FOR PIT RUN, BANK RUN OR STREAMBED GRAVEL

Gravel is placed on the roadway to depth of 6 to 8″, is shaped by a blade grader to proper cross-section and grade. On this is spread approximately 100 pounds per square yard of 1½″ open graded stone or slag which is shaped with sled type drag, rolled and penetrated with 0.5 to 0.7 gallon per square yard of prime tar. Approximately 5 pounds per square yard of fine aggregate is then placed. A second penetration of 0.5 to 0.6 gallon per square yard of heavy cut back asphalt or tar is made and covered with a fine aggregate at the rate of 5 pounds per square yard, all of which is thoroughly compacted and allowed to set up. This results in a smoother surface than the average macadam and is suitable for any type of bituminous wearing surface.

BITUMINOUS STABILIZED GRAVEL BASE

This type of construction and some of the equipment being used is in the experimental stage in this State, as the first projects are now under construction. The projects under construction are stabilizing of 5″ depth of a river gravel surface, using equal length sections of tar asphalt and asphalt emulsion. The sequence of operations may be described briefly as follows:

1. Scarify present gravel surface approximately 5″ depth.
2. Windrow material to center of roadway.
3. Windrowed material is taken by a Barber-Greene traveling plant mixer, bitumen added, also water if necessary, and the combined material is mixed by the pug mill type of design and deposited in a windrow.
4. Windrow is spread to the typical cross-section with patrol graders.
5. Rolling begins with a sheeps foot roller when bituminous mixture is properly aerated and continuous until roller steps out of the base material.
6. Blade grader then shapes surface cutting indentations of sheeps foot roller from surface; then rolling is continued with a pneumatic-tired roller until imprint of tires are not noticeable in the surface.
7. Final rolling completed with flat wheel roller.

From field observations the base material properly compacted is very dense with good dispersion of the bitumen throughout the base material. Sections compacted with the proper liquid content have both maximum stability and density.

This type of construction is economical compared with stone base construction, and if this stabilized base is satisfactory, then there are many miles of road in this State which will probably be constructed of this type.

The new traveling plant mixer is satisfactory and is a valuable piece of equipment which undoubtedly has reduced unit cost of production a considerable amount over old methods of mixing. The pneumatic-tired roller is another piece of new equipment which has given very satisfactory results on this type of construction.

EQUIPMENT

For mixed-in-place and the heavier surface treatment types, the following minimum equipment is required, thereby permitting competitive bids from small contractors:

1. Bituminous distributor with adequate supply tanks.
2. Rubber tired heavy patrol grader.
3. One 10-ton roller.
4. One tank car heater.
5. One sled type smoothing drag.
6. One broom drag.
7. Stone spreading and hauling equipment.

SUMMARY

1. The important changes under bituminous concrete surface are the smoothing of inequalities of the base by patching and the building

of a leveling course to true specification requirements for surface course using the mixed-in-place method, and the placing of wearing surface by use of finishing and tamping machine riding the leveling course.

For the mixed-in-place surface the important improvement has been in the use of open graded aggregate with an increased quantity of bitumen.

2. **Bituminous Concrete:** A much smoother wearing surface for this type has been obtained. There is no appreciable change observed in the density of surface.
   Mixed-in-place: The method described above has produced a uniformly non-skid texture with much higher density by the use and manipulation of fine aggregate of small chips or sand.

3. **Bituminous Concrete:** The saving in cost by present methods of constructing bottom course mixed-in-place has resulted in a saving of $1.00 to $4.00 per ton as evidenced by the competitive bids received. A decided increase is noted in the rate of production; however, the exact per cent of increase cannot be stated.

   Mixed-in-place: There is no apparent change in costs or rate of production noted.

4. The most promising of the above-described improvements is the method of placing bottom course for bituminous concrete surface, resulting in a smoother riding completed surface at a material decrease in cost, thereby releasing funds for the construction of additional mileage.
   Mixed-in-place: The 90 pounds per square yard mixed-in-place surface has been so satisfactory in texture, riding quality, durability and economy of construction that it has now been adopted by this State for the major portion of its new construction. Attached is a sequence of operations for this type as now being used in West Virginia.

THE STATE ROAD COMMISSION OF WEST VIRGINIA

ITEM 23 AMENDED—SPECIAL HEAVY SURFACE TREATMENT — JULY, 1939

90 POUNDS LIMESTONE

Sequence of Operations

New 0.4-0.50 Base Gallon — Prime (Bitumen to be as specified on lans) in two equal applications. Traffic to be prohibited until prime beomes thoroughly set up. After prime is dry, apply:

Surface

1. 0.10-0.15 Gallon — Tack Coat (Same as specified on plans for binder). Cover wet tack coat uniformly with;
2. 75 Pounds — No. 7A Aggregate. Prior to setting up of tack coat, apply;

3. 0.50-0.60 Gallon (Including Sequence No. 5) — Bitumen (Sam as specified on plans for binder). Immediately coat aggregate thoroughly, level and roll lightly. As soon as sufficiently hard to permi spreader equipment cover uniformly with;

4. 5 Pounds — Sand, Broom, then roll lightly, until all sand enter voids, and immediately apply;

5. 0.20-0.25 Gallon — Bitumen (Same as specified on plans for binder) One and one-half foot width, measured one foot from extrem outer edge on surface course and permitting not in excess of one half foot on shoulder. Without delay, apply;

6. 0.20-0.25 Gallon — Bitumen (Same as specified on plans for binder) and cover uniformly at once with;

7. 10 Pounds — Sand. Broom lightly, then roll until sand fills al voids and entire surface becomes thoroughly compacted.

Traffic shall not be permitted on the surface until after completion o all sequences from 1 to 7 inclusive, and bitumen thoroughly set up.

Approved sled runner type smoothing drag or machine, the runners to be not less than 20 feet in length, shall be used for finishing Sequence No. 3.

Graded Requirements for Aggregates

Size No.	1″	¾″	⅜″	No. 4	No. 10	No. 20	No. 50	No. 100
7-A	100	90-100	5-20	0-5				
Sand			100	95-100	75-95	35-75	5-25	0-5

Sand shall meet the requirements of Division 3, Section 23, Standard Specifications, July, 1935.

BITUMINOUS HIGHWAY CONSTRUCTION IN MARYLAND

By

GEO. H. DENT
Asst. Materials Engineer
Maryland Department of Public Works

Maryland, like other Eastern States, has been building roads for many years, and consequently for the last 10 years has been paying the price that a pioneer must pay, in that other states were able to profit by our mistakes. We feel now, however, after a lull that we are again forging to the front.

The Highway System of Maryland is of high order in so far as Secondary roads are concerned, while the Primary roads are out of date. This condition as to the Primary roads is, however, being improved as fast as funds will permit by building dual highways of concrete.

THE DESIGN OF BITUMINOUS HIGHWAYS

1. **Sub-grade:** Only one project has been built in which bituminous material was used to treat the sub-grade. In this connection a bituminous membrane was placed between the micacious silt soil and a gravel sub-base. To date no conclusions have been drawn.

2. **Sub-Base.** No sub-base work involving Bituminous Materials has been done except as noted above.

3. **Base:** Bituminous stabilized base course used in—
   (1) Reconstructing old roads.
   (2) Stabilizing new gravel roads.
   (3) Stabilizing soil as found in sub-grade.

This base course is usually built 5″ thick with a thickened edge. The edge is generally made 3″ thicker than the base. This method eliminates raveling of the edges and presents and preserves a straight line when completed.

The Bituminous materials used have been mostly MC-3 Asphalt and RT-6 Road Tar.

Two projects on the coast have been built utilizing beach sand and RC-2 Asphalt. These projects are 6″ thick without the thickened edges.

The Siebert Stabilometer which is a Triaxial device is used to determine the Bitumen content of the above mentioned types of road. We are not as yet convinced that this is the best method.

4. **Surface:** Bituminous Surfaces of the following types have been built:

(a) Sheet asphalt principally in cities. (b) Hot Asphaltic concrete in cities and on heavily traveled roads. (c) Penetration macadam. (d) Plant mixes Liquefier type. (e) Road Mix Macadam Aggregate. (f) Surface Treatment. Of the above named types, penetration macadam and road mix are the most used.

Penetration macadam generally built on a 5″ dust bound stone base, with a 3″ penetration top using 85-100 penetration asphalt. The top is given two seal coats. First seal coat covered with ¾″ chips, second seal coat with ⅜″ chips.

The Road Mixes are placed 2 to 3″ thick on old roads to improve the riding qualities or on 5″ dust bound stone bases for new roads.

The width of roadway is determined by the following rule:
Less than 250 vehicles daily — 16′.
More than 250, Less than 500 — 18′.
More than 500, Less than 1,000 — 20′.
More than 1,000 — 22′.

The general trend seems to be toward the building of low cost bases, letting these bases serve as a finished roadway for a year or two and then to put on a surface course of some sort.

There have been no marked changes in design during the past year except to make roads wider.

BITUMINOUS PAVING IN VERMONT

By

R. I. ROWELL
Materials Engineer
Vermont Department of Highways

The Vermont Highway Department has adopted and used extensively the "open type" bituminous road mix using crushed stone or crushed gravel screened from 1½″ to ¼″. This material is mixed-in-place with graders of mixed-in-place machines and we strictly limit the percentage of fine stone or pea stone to rather low limits.

We are now experimenting with mixes containing 0-10, 5-15, 20-33% of pea stone. We have stepped up the viscosities of the bituminous materials for this work. We are now using principally RC-4 for asphalt mixes and tar blends composed of 50-65% of our THA-1 tar which shows a float test of 60-150 seconds at 32° C. -- together with 50-35% liquid tar of 20-40 viscosity engler at 40° C. The road mixes, after light rolling, are sanded or keyed with washed coarse sand or pea stone, in the case of Crushed Stone mixes, and then sealed with RC-4 or RC-5 asphalt and the THA-1 tar on the tar projects. Approximately 100 yards of Pea Stone per mile are used for cover.

We are now experimenting with priming treatments on the mixing stone using MC-1. We find that this treatment provides better bond on the stone and enables more thorough mixing with the heavier asphalts. In the case of tar projects, this can be done with a 10 viscosity light tar at 40° C., accomplishing the same purpose. We are also experimenting with the priming of pea stone for seal; piling this material in small stockpiles using 2½ to 3½ gallons of MC-1 per cu. yd.

Vermont being in the glacial region, we have abundant supplies of gravel and we use principally gravel for our base construction. Gravel is comparatively plentiful and our contractors bid as low as 33 cents per yd. on this item. Due to frost conditions encountered, we base 12″ to 24″ on most projects. We have done some experimental work on stabilization of bases with bituminous material. However, these are of too recent origin to draw any conclusions as yet.

CONSTRUCTION OF BITUMINOUS SURFACES IN DELAWARE

By

M. ALLAN WILSON
Testing Engineer
Delaware State Highway Department

BITUMINOUS CONCRETE

We in Deleware place bituminous concrete surface course on two different types of base; the most widely used is 6″ of cement concrete pavement upon which is placed two inches of bituminous surface course. This two-inch bituminous surface course is divided into 1¾″ base courses with approximately ½″ compacted top dressing. The second way in which bituminous concrete is placed is on the water-bound macadam base course from six to eight inches in thickness. We are fortunate in that we do not usually have to stabilize any of the subgrade under either of these two types of pavements; however, should it be necessary to stabilize either type we would select, and are able to obtain, an A-1 classification soil which would be placed approximately 12 inches deep and compacted with a sheeps-foot roller. The material contained as stabilized layer would not have a plasticity index greater than six.

The specifications under which cement concrete base course is placed is, in all practical purposes, the same specification under which cement concrete pavement is placed. We require dummy joints in the base course every thirty feet and further require an auto-clave test not to exceed 5/10 of one per cent. The material placed as base course conforms to the general requirements of our specifications.

In general the base course is made up of material graded on the 1¼″, 1″, ¾″, ½″ and No. 4 sieves. We require a job mix formula to be submitted from which we select or direct that type material as to gradation, amount of liquefier, amount of asphalt and amount of filler to be placed. The surface course is placed under a specification requiring the material to be evenly graded on the ½″, ⅜″, No. 4, No. 8 and No. 20 sieves. This material is also placed under a job formula as is the base course. The asphalt content of the base would ordinarily run approximately 3.5% with the liquefier approximately 5/10 per cent, while on the surface course the asphalt content would average 5.4% while the liquefier averages 7/10 per cent.

This type is handled and spread and further rolled in the customary manner used by most of the states.

2. We do an extensive amount of bituminous surface treatment work which is done in the following manner:

A contract is usually let two or three years prior to the surface treatment, which includes the grading, ditching, general shaping and placing of traffic bound slag or stone, which at the end of three years gives us a good traffic-bound roadway ready for surface treatment.

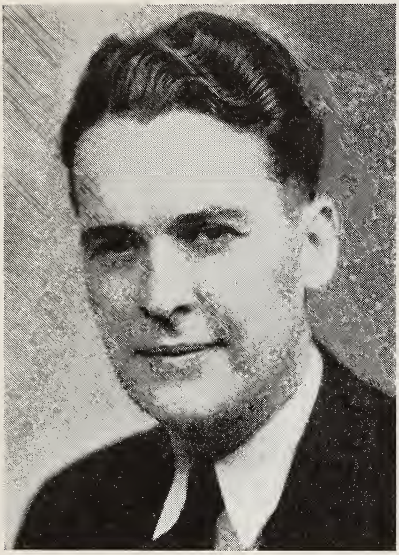
Upon the roadway is placed on an average of ½ gallon of prime and twenty pounds of stone chips per square yard. Upon this is placed ¼ gallon of seal and 25 pounds of stone chips over the same square yard. The material as used generally conforms to an RC-1A and RC-2 of both prime and seal respectively.

Prior to the material now being used we used Mexican Petroleum asphalt practically exclusively and are fortunate in obtaining excellent results; however, this RC-1 and RC-2 has so far given us very good results comparatively speaking.

Present Practice in Idaho for the Design and Constuction of Highway Surfaces

By

CLIFF HALLVIK
Materials Engineer
Idaho Bureau of Highways



This article is a brief summary of the present accepted principles involved in the design construction, and bituminous treatment of highways in Idaho.

1. SELECTION AND COMBINATION OF MATERIALS FOR SUBGRADE, SUB-BASE, BASE AND SURFACE

A. Subgrade

Highway subgrade design in Idaho may be divided into two general classifications: (1) Designs in rolling and mountainous country; (2) Designs in flat valley or plateau areas. In the former the balanced cut and fill design is used. In this case we are not particularly selective in the quality or type of materials used, as our grading section, almost exclusively, is built from the materials adjacent to the line. We use every type encountered with the possible exception of the A-8 group. We rely for stability upon the proper construction of the grade, suitable drainage, and the placing of selected granular materials (sands, gravels or crushed rock) at depths varying from 3″ or 4″ base courses up to a maximum of 30″. This stabilizing material is placed in layers and spread from shoulder to shoulder on a crowned subgrade section.

In (2), design and construction over flat terrain, the line to be traversed may often be in low or irrigated country where peats or other organic materials are encountered. In this instance, the grade line is laid several feet above the existing ground line and the subgrade is constructed of selected materials hauled in from acceptable borrow sources.

In all design and construction, special care is taken to remove questionable materials at 0.0 grade points in passing from cut to fill. Also, since the advent of our soils investigation and the making of soils profiles, design takes care of the cross-hauling of undesirable materials from the top layers of cuts to the lower portions of the adjoining fills.

Most of our state is particularly blessed with an abundance of sands, gravels or rock ledges located within practical haul distances of a project.

The abundance and proximity of these materials has caused us to look toward "bridging over" our soil subgrades for wheel load support and stability of our highway road beds.

The choice of selected borrow materials in cases wherein the fill is to be made entirely or in part from such sources, is made on the basis of comparative laboratory test results and the availability of materials.

Selected Subgrade Materials

We divide our selected subgrade materials into two classes, namely: (a) Gravel or rock for select borrow, and (b), soil for subgrade borrow. The division is made on the basis of the soils definition as given in AASHO M 57-38, "as a material having more than 35% passing a No. 200 mesh sieve."

A survey is made of the area to locate probable sources when it is found necessary to import selected borrow materials. Prospective sources are investigated by test pits and auger borings. Samples are submitted to the laboratory and the following tests are made:

Gravel or Rock Borrow:

1. Grading.
2. California Bearing and Swell.
3. Wear (Los Angeles).
4. F. M. E. AASHO T-93-38.
5. Liquid Limit AASHO T-89-38.
6. Plasticity Index AASHO T-91-34.

Soils for Subgrade Borrow:

1. Grading.
2. Complete Atterburg Constants.
3. Proctor Analysis.
4. California Bearing and Swell.

Idaho does not have any standard specification limits which apply to the qualifying or disqualifying of selected sub-grade materials. However, sources are compared on the basis of the above tests and chosen or selected from the best source consistant with cost. In cases where only questionable materials can be had within reasonable cost limits, we choose the best and then rely upon adidtional subbase or base material to provide the proper stability of the roadway surface.

We are inclined to choose a soil on the basis of the Atterburg Tests and use the California Bearing and Swell values as substantiating information. On gravel or rock borrow the grading, and the California Bearing and Swell Test are given preference; the Liquid Limit and Plasticity Index tests are used as the substantiating values. We designate a particular source and rely upon the Engineer to make proper selection of the materials from that source, instead of setting up specifications on selected materials.

B. Sub-Base

Selection of sub-base material is made according to definite specification limits in Idaho. In our choice of a sub-base material we again make a thorough field survey of probable sources and the samples are submitted to the laboratory. The same tests are made as stated for Selected Gravel or Rock Borrow. In this case, however, the following definite specification requirements must be fulfilled:

Grading	3″ Max.	2″ Max.
Passing 3″ Sq. Screen	95-100%	
Passing 2″ Sq. Screen		95-100%
Passing 1″ Sq. Screen	40- 75%	40- 80%
Passing ¼″ Sq. Screen	25- 55%	25- 55%
Passing 200 Mesh Sieve	12% Max.	12% Max.

We specify screening or crushing, preferably the latter, in cases in which the materials proposed cannot meet the above specifications as pit run.

The following more or less generally recommended control values for Liquid Limit and Plasticity Index are set forth in our specifications for subbase materials: "If the material passing the No. 40 sieve does not exceed 20% of the total, it shall have a plasticity index of not to exceed eight (8) and shall have a liquid limit of not to exceed thirty (30), except that a material having no plasticity limit may have a liquid limit of not more than thirty-five (35). If the material passing the No. 40 sieve exceeds 20% of the total, it shall have a plasticity index of not to exceed six (6) and shall have a liquid limit of not to exceed twenty-five (25)."

Our recommended specification limit for wear for sub-base materials, when tested in accordance with A.A.S.H.O. T-96-38 with the Los Angeles Abrasion machine, is fifty (50) per cent.

There are no specified limits for the California Bearing and Swell test values in Idaho. The test results are used as a guide in the selection of a source. The values used by California serve as our criterion as to the quality of the source under investigation. We have found that material showing excessive swell will ordinarily have a liquid limit or plasticity index in excess of that provided in our specifications.

C., D. Base or Surfacing Materials

Our selection of aggregate to be used for base or surfacing materials is made in a manner similar to the selection of sub-base materials. However, our standard specification limits are somewhat different, as follows:

Grading	Crushed Gravel	Crushed Rock
Passing 1″ Sq. Screen	100%	100%
Passing ¼″ Sq. Screen	45- 65%	40- 60%
Passing No. 10 sieve	25- 45%	15- 35%
Passing No. 200 sieve	12% Max.	12% Max.

There is a trend to specifying ¾″ maximum size in some portions of the state to insure a more uniform grading of the minus ¼″ material.

On all surfacing materials our specifications provide that the portion passing the No. 200 sieve must be less than 65% of the minus No. 40 fraction. The specifications on the Liquid Limit and Plasticity Index are the same as provided for gravel or rock sub-base.

Proposed sources of base or surfacing materials in the vicinity of the project are thoroughly investigated and samples are submitted to the laboratory for the following tests:

1. Mechanical Analysis.
    (a) The original grading is reported and then the over size in the sample is crushed in a small laboratory crusher to simulate field crushing and produce a grading as required by our standard specifications.
2. Weight per cubic foot compacted.
    (a) This value is used by the design department in setting up the tonnage of base or surfacing required on the project.
3. Cementing Value.
    (a) California cube compression test.
4. Specific Gravity.
5. Absorption.
6. Liquid Limit, (AASHO T-89-38).
    (a) Made on combined crushed and natural fines.
7. Plasticity Index.
    (a) Determined on combined crushed and natural fines.
8. Field Moisture Equivalent.
    (a) Determined on combined crusher and natural fines.
9. Lineal Shrinkage (Rose Method).
    (a) Determined on combined crusher and natural fines.
10. Per cent wear, Los Angeles, (AASHO T-96-38).

Tests are made in the laboratory combining the proposed sources of filler with the base or surfacing materials, when it is obvious that the material to be produced will require earth or sand filler. Our specifications for base or surfacing materials are binder or filler included specifications, that is, the material in the finished road surface must be within the specification limits.

We recommend 40 per cent wear as the upper limit for base and surfacing materials, though we have not as yet incorporated a specification for per cent wear.

1. Aggregate for oiling: Our materials for oiling come from either glacial or water deposited gravel deposits or rock quarries predominately composed of basalt, shales or quartzites. In general, any crushed or screened materials that meet all the specification requirements for bases or surfacing are considered acceptable as aggregate for oiling. Preliminary samples are submitted to the laboratory from the proposed sources and after the

above tests for surfacing materials have been completed, the aggregate is tested for oil requirements, stability and swell. Ordinarily, an oil of the SC-3 grade is used for this purpose. Oil requirements are determined by use of the California Bitumen Index Charts. Three or four laboratory mixes are made using different curves from the chart. The completed and cured mixes are then tested for stability in the Hveem Stabilometer. Materials indicating excessive oil demands or a stability of less than 35% are considered unsatisfactory. The swell test is made in accordance with AASHO T-101-38. Materials showing an excessive per cent of swell are not approved for use.

2. Liquid Asphalts: We are guided chiefly by past experience in the selection of the grade of liquid asphalt to be used in an oil mat. We use SC-3, for road mix projects of the dense graded type. Our oiling season is short in most of the state and much of our work is done when the season is well advanced and cool weather has set in. We find that SC-3 oil lends itself more readily to thorough mixing and handling under the conditions prevailing in this area. For more open graded mixes and when oiling is to be done in reasonably warm weather, we recommend the use of the SC-4 grade. We often recommend an alternate of SC-3 or SC-4 to be placed in the contract. The final selection of the grade will be dependent upon conditions at the time of construction.

One of the reasons for recommending the alternate is the increased use of the traveling plant for mixing. We favor the SC-4 grade for this type of construction.

We recommend the use of MC-3 grade liquid asphalt for stationary plant mix projects in the majority of cases. However, we have some successful projects mixed with MC-4, RC-3, RC-4 and SC-4. At present we are experimenting with SC-6 on a plant mix project in southeastern Idaho. The project is now under contract but oiling operations have not yet commenced.

3. Cover Coat Materials: Aggregates for cover coat may be screened or crushed gravel, or crushed rock. Our standard grading specifications are:

Passing ½″ Sq. Screen	100%
Passing 10 Mesh Sieve	8% Max.
Passing 200 Mesh Sieve	4% Max.

Materials must have a per cent of wear less than 40 when tested with the Los Angeles Abrasion machine. Lower percentages are preferable. Aside from the wear characteristics of an aggregate proposed for cover coat material, our selection is also influenced by results obtained from the asphalt adhesion test. At present we employ a modified Dow Stripping test, using RC-4 Liquid Asphalt. Materials showing excessive stripping are not recommended for use.

Occasionally, our standard specifications for cover coat materials will be changed to handle some local condition on certain projects. Perhaps a ¾″ or ⅝″ maximum size will be specified. A sand cover material is often used to seal old mats by maintenance forces, thereby using cheaper local materials.

There is a trend toward the use of the larger size cover rock (¾″ or ⅝″ Max.) and the use of heavier asphaltic materials for seal purposes.

4. Liquid Asphalts for Cover Coat: Liquid asphalt for cover coat is generally specified as MC-4 with RC-4 as an alternate. RC-4 seems to be favored and good results have been obtained using this grade. Best results have been obtained with use of an SC-3 or SC-4 grade of liquid asphalt when sand is to be used for sealing.

We are experimenting with the use of heavier asphalts on a few projects this year. In one case we have specified the use of a 151-200 penetration asphalt cement. On two other projects we have specified the use of SC-6 liquid asphalt. Comments as to the success of this treatment cannot be made at this time.

2. DETERMINATION OF THICKNESS OF SUBBASE, BASE AND SURFACE FOR SUBGRADE

The determination of the thickness of subbase, base and surfacing for a design has not as yet been reduced to any simple formula. Past experience is still perhaps the controlling factor when recommending the amount of ballast the grade shall have. However, this year we have taken definite steps to follow, at least for a guide, the thicknesses suggested by the U.S.B.P.R., (Dist. No. 2) as a result of physical soil tests and gradings; California by the use of the Bearing and Swell test; and Arizona as a result of experience with the shrinkage test.

We have divided our design into three general groups in an attempt to intelligently determine the thickness of sub-base, base and surface required on our highways; (1) arid sections, (2) arable sections and (3) sections where the water table is near the grade line. For class one, we are inclined to favor the minimum thickness in the tables below, while for classes two and three, we lean toward the maximum, tempered by past experience.

(A) District No. 2 of the U.S.B.P.R. Table of recommended depths of selected materials for Stabilization: (Result of test sections and observation).

Mechanical Anaylsis of total sample	Excellent A	B	Good A	B	C	D	Fair A	B	Poor A	B
% Passing No. 10	25–65	25–65								
% Passing No. 40	10–50	10–50								
% Passing No. 200	25–	15–	15–	15–	35–	35+	35–	35+	35–	35+
Liquid Limit	25–	35–	35–	35+	35–	25–	35–	50–	35–	50+
Plasticity Index	6–	6–	15–	6–	10–	6–	30–	20–	30+	20+
% Passing No. 40	10–50	10–50								
Group	A 1, 2, 3		A 2, 3, 4, 5				A 2, 4, 5, 6, 7		A 5, 6, 7	
Thickness of sub-base, base and surfacing	0 to 6 in. Average 3 in.		2 to 8 in. Average 6 in.				9 to 18 in. Average 12 in.		12 to 24 in. Average 15 in.	

(B) California method of surface stabilization based on bearing and swell values of subgrade soils as developed by tests and experience.

Bearing Value Per cent of Standard	Swell	Total thickness of sub-base, base and surface
50%+	0% - 2%	Wearing course only
30%+	0% - 3%	5″ Total
20%+	0% - 4%	6″ Total
10%+	6% -	9″ Total
5%+	6% -	15″ Total
3%+	6% -	18″ Total

Note: If the bearing value carries with it a higher percentage of swell than is shown in the table, we consider the use of the next greater thickness of subbase, base and surfacing.

(C) Arizona Method based on Lineal +Shrinkage values.

Lineal Shrinkage	Total thicknes sof sub-base, base and surface
0% - 2%	Wearing course only
2% - 4%	6″ Total
4% - 6%	8″ Total
6% - 9%	12″ Total
9%+	15″+ but not to exceed 24″ total

Note: Arizona has recently prepared a new table based on Lineal Shrinkage, Centrifuge Moisture Equivalent, Plasticity Index and Grading. We have not as yet considered this new table in preference to the one above.

In using the above tables for a guide in the determination of the necessary sub-base, base and surfacing required for a design, we approach the problem in the following manner: If the subgrade sample has less than 35% of the total passing a No. 200 mesh sieve, we use Table (B) as our criterion, and Tables (A) and (C) as supporting evidence. If the subgrade sample has more than 35% of the total passing a No. 200 mesh sieve, we use table (A) as our criterion and tables (B) and (C) as supporting evidence. The values thus determined, together with our knowledge of past performance in the vicinity of the proposed project, forms the basis of the final decision as to the total thickness of the sub-base, base and surface.

3. CHANGES IN SPECIFICATIONS FOR AGGREGATES AND ASPHALTIC MATERIALS

Prior to 1938, Idaho's general specification for grading of aggregates to be used in oil mixes was substantially as follows:

Passing 1″ circular or ⅞″ sq. screen	100%
Passing ¼″ square screen	45 - 55%

That portion of the material passing the ¼″ screen shall be well graded from coarse to fine without not over 15% passing the 200 mesh sieve.

The above specification was used until the adoption of the grading specification noted previously in this paper. As long as the above grading was obtained and the material came from a source showing suitable qualities as indicated by a wear test no other requirements were necessary. Materials meeting these specifications were often placed in the base or surfacing to be used in the construction of an oil mat at some future date. In order to set up this material, soil fines termed "binder" were added in amounts as necessary to set up the base or surfacing. The qualities of these fines or binder material were not seriously considered. When funds became available, perhaps months or years later, the surface was scarified to produce the necessary material for an oil mat. The net results was that the specifications applying to the aggregates for oiling lost their identity, and that gravel and soil mixtures of unknown compositions were incorporated into the oil mat. This practice is no longer followed in Idaho except in very rare instances.

Liquid Asphalts

Idaho's specifications for this material follow those outlined by the Asphalt Institute with the exception of a paraffin scale requirement on liquid asphalts. We feel that these are merely identification tests and that there is a definite need for quality tests that may predict the performance of the material.

METHODS OF CONTROLLING CONSTITUENTS AND PROPORTIONING ASPHALT — AGGREGATE COMBINATIONS

This discussion will be limited to the proportioning of liquid asphalts and aggregates as encountered in the ordinary investigation of road mix and plant mix projects using a dense graded aggregate. Preliminary investigations of gravel or rock deposits are made in the field for the selection of suitable sources of aggregates as previously discussed. Samples are submitted to the laboratory and are prepared to simulate construction crushing or production from the source under investigation. Materials must comply with the specifications as previously mentioned in this paper. The percentages of oil (usually using an SC-3) are determined by use of the "Bitumen Index" chart originated by California. A set of three or four specimens are made up using as many different curves for oil content from the chart. The curves used are those which give a sufficient amount of liquid asphalt to produce the correct color workability, and consistency as judged by the test operator. Each of the mixtures is subjected to test for stability using the Hveem Stabilometer and the recommended Hveem procedures. Mixtures showing a stability of 35% or greater are considered acceptable. Mixtures showing a percentage of less than 30% are not recommended for use.

Swell tests in accordance with A.A.S.H.O. T-101-38 are run on the minus 10 fraction of the mixtures. We have a tentative limit of 1½% maximum swell for most of the aggregates that may be considered for use as oil aggregate.

Further selection is based on the operator's judgment if the mixture successfully passes the above tests. We do not choose the highest stability as indicated by the stabilometer, but choose the mixture giving desirable stability with an oil content high enough for workability and future durability of the mix.

The percentages of oil recommended by the laboratory are used by the designing engineer in setting up the oil quantities for the estimate, and serves as a guide for the engineer in the field as to the quality of liquid asphalt that may be required for the mix. We have found the oil demand for aggregates to vary from 5½% for some volcanic scorias and tufts to as low as 2½% for some quartzites, each mixture meeting the same grading specifications. Throughout this range, the laboratory has been able to predict the oil requirements on projects constructed the past two seasons with an encouraging amount of accuracy.

We encourage the use of the California Bitumen Index Chart on all oil projects to be used as a guide for the engineer. Final decision as to the oil content to be used in the mix in the field is still left to the judgment and experience of the oiling engineer.

5. SOIL SURVEYS IN IDAHO

Soil studies were started in this state about 1933 or 1934. This covered miscellaneous sampling and testing of soils from spring break ups and occasional sample taken along a proposed project to classify the soil and use the results to some extent in the determination of the design. Our first complete soil survey was made on an experimental project in 1936. However, it was not until the beginning of the current year that soil surveys and studies became a part of our preliminary work on all projects and therefore procedure is relatively new in Idaho. However, we have adopted definite procedures, tests and applications which may be outlined here.

6. DEVELOPMENTS IN THE USE OF SOIL SURVEYS

The subgrade survey is made according to procedures outlined in A.A.S.H.O. Method T-86-34 by the District Materials Engineer, Notekeeper, and untrained assistants, with the cooperation of the Soils Engineer.

When the borings are completed, 50 pound samples of the various layers encountered are secured at intervals not exceeding one-half mile and shipped to the laboratory for analysis. The following tests are run on all samples, however in the near future, we hope to have sufficient data to correlate out results and eliminate a portion of the tests:

1.	Liquid Limit	A.A.S.H.O. Method T-89-38
2.	Plastic Limit	A.A.S.H.O. Method T-90-38
3.	Plasticity Index	A.A.S.H.O. Method T-91-38
4.	Shrinkage Limit	A.A.S.H.O. Method T-92-38
5.	Field Moist. Equiv.	A.A.S.H.O. Method T-93-38
6.	Centrifuge Moist. Equiv.	A.A.S.H.O. Method T-94-38
7.	Lineal Shrinkage	Rose Method run on −10 M. Mat'l.
8.	Specific Gravity	R. R. Proctor Method—E.N.R. 8/1/33
9.	Bearing Value and Swell,	California Method.
10.	Mechanical Analysis	A.A.S.H.O. Method T-99-38 except that hydrometer readings are completed at the end of two hours and the resulting curve extrapolated for the colloid fraction.

11. Compaction and Density — A.A.S.H.O. Method T-99-38 except that U. S. Bureau of Reclamation mold of 1/20 cu. ft. capacity is used and the test run on –¼″ material.

12. Proctor Plasticity Test run in conjunction with compaction test.

Lineal Shrinkage is not computed according to A.A.S.H.O. method T-92-38 because we were unable to check computed values with values obtained by the Rose method after a series of experiments on Idaho soils run on both the –40 and the –10 fraction.

Specific gravities are run according to the method described in E.N.R. of 8/1/33 by R. R. Proctor because at the time soil studies were started in our laboratory, A.A.S.H.O. had no specified method for soils. The Proctor method has proven adequate to fulfill our needs.

Hydrometer readings are stopped at the end of two hours to shorten the time required for the test and thereby increase production. We do not feel that this procedure renders our results impractical because we obtained readings well into the clay fraction (which includes the colloids) and because we have attempted no soil stabilization utilizing chemicals wherein we would have to consider the electro-chemical phenomena of adsorption and base exchange, involving the colloidal phenomena. We feel, therefore, that the approximate determination of the colloid fraction obtained by means of extrapolating the grading curve is sufficiently accurate for our needs.

Completed test reports are then sent to the District Materials Engineer who completes the profile and returns it, together with his recommendations for stabilization, to the laboratory for checking, further recommendations, and printing.

Recommendations include thickness of subbase, base, and wearing course, determined as previously described; sub-drainage if required; wasting of unsuitable material; economical disposition in the bottom of fills of material of poorer quality; recompaction of soil through cuts that do not meet A.A.S.H.O. density specifications; sprinkling and compaction requirements.

Recommendations are then followed through the designing Engineer and incorporated into the plans and specifications.

7. SUMMARY AND MISCELLANEOUS

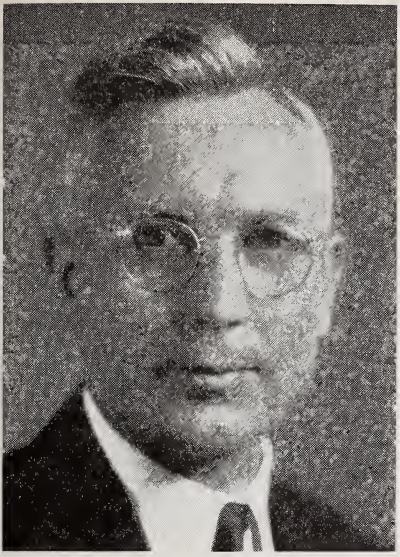
Progress in the selection of material for subgrade, sub-base, base and surface has been made in the past few years. Tests of materials for subgrade control our design and construction methods. New specifications for the selection and control, the grading and quality of subbase, base and surface materials, have brought about more headaches for the materials department, but definitely has lead to better construction.

The use of the methods outlined in the discussion as a guide for determining the thickness of subbase, base and surface is a step out of the dark. Thickness of subbase, base and surfacing cannot be determined with exactness because of the many variables entering into its determination. However, studies as are made by the B.P.R., California, and Arizona are a step toward the solution of the problem. Future studies of the projects we are now building should further aid us in this problem because of the soils work we are now doing prior to construction.

Better control, by specifications, of the grading and quality of aggregates for oiling is producing better and more durable surfaces. However, little or no developments have been made in the selection of asphaltic materials from a quality stand-point. For the most part the standard specifications for an Asphaltic Material are nothing more than requirements to identify the liquid asphalt as to grade. There is definite need for tests and specification covering this material from a quality standpoint. Many states select their asphaltic materials on the basis of the Oliensis Test. In Idaho we have not chosen to accept this test as a measure of quality for reasons of past performance of both negative and positive spot materials and our location with respect to sources of Liquid Asphalts.

In Idaho we are now discarding the ''Thickened Edge'' in the construction of bituminous mats when more than 20 feet in width. Base and surface courses are built shoulder to shoulder and therefore we do not feel it necessary to thicken the edges of the oil mat, choosing to make the mat an additional foot wider for the same quantities of materials.

Bituminous Highways in Utah



By
H. S. CARTER
Professor of Highway Engineering
Utah State Agricultural College

and
LEVI MUIR
Materials Engineer
Utah State Road Commission

1. Accepted principles of design of bituminous highways.
   - A. Selection and combination of materials for:
     1. Subgrade. The soil is classified. Poor soils are replaced with good material or the sub-base is increased in thickness. One case to three feet.
     2. Sub-base. The sub-base is generally designed on the basis of experience and W. S. Housel's formulas for the thickness of flexible pavements.
     3. Base. A six-inch base of three-inch maximum material is used unless subgrade soils require such treatment as indicated under (1) and (2) above.
     4. Surface. Main highways, 2½ inches thick.
        Secondary highways, 1½ inches thick.
2. Changes, improvements and trends within the past year.
   - A. Changes in specifications or tests for aggregates and asphaltic materials.
     The liquid limit for base course aggregate has been lowered from thirty to twenty-five.
     The liquid limit remains at thirty for surface gravel.
     The plastic index has been lowered from eight to six for both top course surfacing gravel and sub-base gravel.

B. Changes in methods of proportioning and controlling constituents in asphalt aggregate combinations of all types.

No changes have been from past practice.

A formula is used for close graded aggregate for the determination of voids in the compacted dry aggregates. Allowance is made for moisture and for oil absorption. Mixtures are prepared in the laboratory and observed under heat tests to determine the oil absorption factor.

C. Developments in the use of soil surveys.

Soil surveys are now made on all projects. The results are used to:

1. Classify the soil
2. Cull out poor soils
3. Determine side slopes in cuts and fills
4. Determine height of fills
5. Stabilization of subgrade
6. Check soils under fills for possible subsidence
7. Plan the drainage of the highway
8. Determine the shrinkage factor.

Soil survey results are supplied to the design department. It is anticipated that soil profile of the project will be provided to the contractor.

D. Changes in specifications for, or design of, subgrade, sub-base, or base courses.

1. Fills are compacted in eight inch layers with a sheeps foot roller. Soil at optimum moisture content as determined by the Proctor test.
2. Sub-base gravel and surface gravel cover the entire width of the road grade.

E. Progress reports on existing experimental projects. None.

F. Trends

A. Utah is just beginning to use the negative spot tests for D and E grade asphalts, and SC 6 and RC 4 road oils.

B. Lighter products such as SC-6 which have a negative spot test are now used where formerly D grade asphalt was used.

C. There are no stabilization projects in progress in Utah at the present time. Some short projects are anticipated within a year using:

1. Emulsified asphalts direct with natural soil.
2. Emulsified asphalts with gravel and natural soil.
3. Lignin with gravel and natural soil.
4. Gravel and natural soil. This combination is to be the control mix.

D. Sealing is now done with RC-4 with stone chips ⅛″ to ⅜″. Formerly an MC-4 was used.

Aggregates for Seal on Bituminous Mats and Bituminous Surface Treatment on Stabilized Bases

By

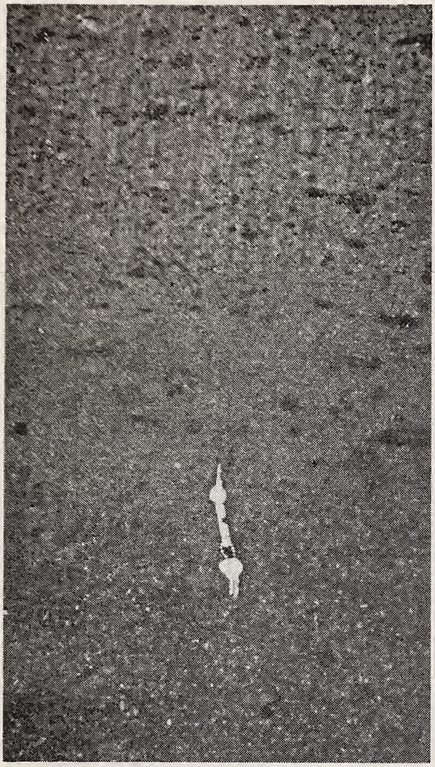
L. A. FRENCH
Engineer of Tests
North Dakota State Highway Department

North Dakota has constructed approximately one thousand miles of bituminous surfaces to date. These surfaces are either 1½ or 2½ inches thick. As in most all bituminous construction sealing the surface has been standard practice. Our first specifications covering the sand for seal coating were very indefinite, the specifications stating "... that a coarse sand shall be applied in a sufficient quantity to take up any excess bituminous material ..." These specifications remained in force until 1937. At that time a change was made requiring a sand which passed the one-fourth inch sieve and not more than 20 per cent passing the No. 10 sieve. In 1938 the sand used primarily to blot up excess seal coat oil was discarded and a new specification using aggregate for seal was adopted. This idea which I believe originated from the stone chip treatment has been used entirely on all our newly constructed bituminous surfaces and this year is being extended to use as a wearing course and surface treatment directly on stabilized bases on light traffic roads. It is also being used for the first time this year as seal aggregate treatment on old bituminous mats which have been down for several years.

AGGREGATES FOR SEAL ON BITUMINOUS MATS

The specifications covering the seal aggregate for bituminous mats have gone through several changes during the past year. The new specifications covering this material require 100 per cent passing the one-half inch sieve, 20 to 50 per cent passing the one-fourth inch sieve and 0 to 10 per cent passing the No. 8 sieve. Crushing is required on all material retained on the one-half inch sieve. The aggregate shall not contain more than 3 per cent moisture when applied.

The finished bituminous mat is opened to traffic for at least two weeks or until a smooth, well-compacted surface has been obtained. All unsatisfactory areas must be repaired before the cover aggregate is applied. The surface is broomed and RC-2, RC-3, or RC-4 is applied at the rate of 0.4 of a gallon per square yard. When the desired stage of tackiness has been attained the cover aggregate conforming to the above specifications is spread at the rate of 21 pounds to the square yard.



Old Type Sand on Bituminous Mat

Seal Aggregate on Bituminous Mat
Dark Line is Center Stripe

View of Project — Finished with Seal Aggregate

Immediately after spreading, the covered surface is rolled with a oller weighing not less than eight nor more than ten tons. The initial olling is done with pneumatic tired rollers followed immediately by mooth steel rollers which continue rolling until the desired surface finish s attained.

The armor coat applied in this manner gives a hard, smooth riding urface. It has very good skid-proof qualities. It also has the distinct dvantage of giving a very light colored traffic surface which is excellent for night driving and answers the many complaints we have had regarding the difficulty of night driving on black top pavements. We have used RC-3 to center stripe these light colored surfaces.

AGGREGATES FOR SINGLE AND DOUBLE BITUMINOUS SURFACE TREATMENT ON STABILIZED BASES

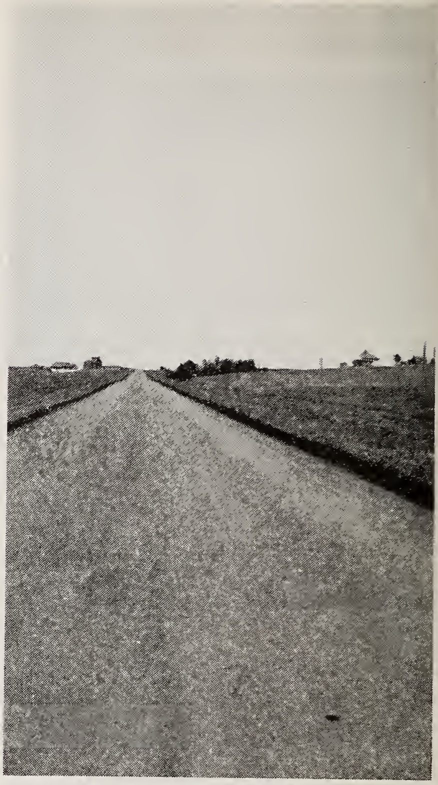
We have many miles of road in North Dakota where the amount and kind of traffic do not warrants the building of bituminous mats of the dense graded aggregate type. However, since the original gravel placed on these roads has largely disappeared it has been necessary to rebuild them to a certain extent. We have added additional gravel and soil binder in the amount necessary to give us four inch stabilized bases. To give a smooth-riding, dustless surface we have drawn up specifications covering a single and double bituminous surface treatment to be placed directly on the stabilized base. The material used for cover aggregate shall be composed of hard durable material, shall not contain more than 3 per cent moisture by weight when applied and shall conform to the following gradations:

	Percentage by weight passing	
	Grade 1	Grade 2
¾″	100	
½″		100
¼″	15 - 45	0 - 40
No. 8	0 - 10	0 - 10

In the single type bituminous treatment 0.3 to 0.4 of a gallon per square yard of SC-2 is applied to the stabilized base. Sufficient time is allowed for proper penetration and hardening of the prime coat. The second application consists of RC-4 applied at the rate of .22 to 0.3 of a gallon per square yard. Cover aggregate conforming to specifications for Grade 1 is applied immediately at the rate of 22 pounds per square yard followed by initial rolling with pneumatic rollers. Final rolling is done with smooth steel rollers. If it is necessary, in order to secure an even distribution of the aggregate, the surface is broomed between trips of the roller. This alternate rolling and brooming is continued until the cover aggregate is embedded and the surface thoroughly compacted and uniform in texture.

Methods used in constructing the double type bituminous surface treatment require the application of 0.3 of a gallon per square yard of SC-3 to the stabilized base followed immediately by spreading 21 pounds to the square yard of Grade 1 Cover Aggregate which is rolled and main-

Grade 1 Cover Aggregate on Stabilized Gravel Base, Single Treatment



Single Type Bituminous Surface Treatment. Grade 1 Cover Aggregate on Stabilized Gravel Base

Applying SC-3 on Stabilized Sand Base
Double Type Bituminous Surface Treatment

tained in the same manner as the single treatment until firmly compacted. RC-4 is then applied at the rate of 0.2 to 0.25 of a gallon per square yard followed immediately by spreading 21 pounds to the square yard of grade 2 Cover Aggregate which is rolled and maintained until the surface is well compacted and uniform in texture.

AGGREGATE FOR SEAL ON OLD BITUMINOUS MATS

During the present construction season the Highway Department intends to apply seal aggregate to several miles of old bituminous surfaces. The specifications covering this type of aggregate are as follows:

Passing ½″ sieve	100%
Passing ¼″ sieve	20% to 70%
Passing No. 8 sieve	0% to 15%

Applying Seal Aggregate on Old Bituminous Mat

Prior to spreading the seal aggregate the old bituminous surface will be repaired. The repairs on the bituminous surface will consist of patching all small breaks and scarifying, remixing, and relaying larger sections where necessary. The entire surface will then be swept to remove all debris and loose material. These surfaces were sealed originally and in some cases once or twice since construction, therefore careful field control of the amount of cut back per square yard will be necessary. The basis of estimate was .22 of a gallon per square yard but it is anticipated that .15 to .18 of a gallon per square yard of RC-4 will be sufficient. At the time of application the RC-4 will be heated to the highest possible temperature consistant with good practice. Application of the aggregate will follow immediately at the rate of 18 pounds to the square yard. The initial rolling will be with a pnuematic tired roller followed by smooth steel rollers until the surface is considered properly compacted and uniform in texture.

CONCLUSION

The statement in the opening paragraph of this report regarding the origination of this idea from the stone chip treatment may be misleading. In regard to the type of material we are using the aggregates used as a rule contain from 20 to 50 per cent fractured pieces. However, on one project where the gravel pit runs heavy to rock the contractor is diverting a portion of the oversize as he produces his material for the stabilized base, and crushing this oversize for cover aggregate. In this case we have 100 per cent fractured particles. The results obtained on newly constructed bituminous mats and surface treatments on stabilized bases have been very satisfactory.

The writer wishes to acknowledge the asistance of the Construction Department in furnishing information regarding methods of application

Design and Construction of Bituminous Surfaces in Michigan

By

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There have been no marked changes with respect to the design of bituminous highways in this area during the past year. Although not confined to the past twelve months, there has developed a greater and more practical appreciation of the necessity of adequate foundations. In this region the State, counties and certain of the larger cities are systematically preparing bases by stabilization or reconstruction for subsequent bituminous surfacing.

Of bituminous surfaces, the graded aggregate type, generally referred to as "oil aggregate," is rapidly increasing in popularity. The apparent success of the type in this state has aroused the interest of neighboring states. Many counties have constructed satisfactory surfaces by road mix methods for a number of years and still continue to do so. However, the Michigan State Highway Department is committed to plant-mixed and machine-laid surfaces and is constructing many miles of them. On a very large WPA street improvement project in Detroit the same is the case. Several large contractors are prepared to ship such mixtures long distances. Thus, on contract work, plant mix is displacing road mix in popularity.

The reasons for this are greater independence of weather conditions and the possibility of producing better and more uniform mixtures. Gradations of natural aggregates are specified and combinations are often resorted to. Manufactured fillers are generally added. Slow curing oils of somewhat heavier grade than the No. 4 grade are most often used. Materials are approved in advance and trained inspectors supervise both the mixing and laying operations.

There has been a tendency to at least experiment with very viscous oils, which, if carried somewhat further, would result in the oil aggregate becoming a true asphaltic concrete.

Thus far seal coats have not been applied to the very dense mixtures laid by the State, and it is reported that the necessity for them is not apparent. Counties frequently seal with RC or emulsion where farm traffic is a factor.

The apparent adaptibility of the type has lead to its experimental application as a resurfacing material to old concrete or bituminous pavements. Reports indicate that it may find a field of usefulness here.

SECTION III

Design and Construction of Bituminous Surfaces

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Review and Summary of Papers Prepared On Design and Construction of Bituminous Surfaces

By

C. F. SEIFRIED
Office Engineer
Wyoming State Highway Department

The development in the design and construction of low cost bituminous surfaces has been forced by the necessity of giving the traveling public in a very short time a smooth, dustless, all-weather surface. Necessity, at first, caused highway engineers to give this surface without the aid of scientific research.

The top, or surface of the structure, was, naturally, given first thought. Afterwards, we worked downward from the surface. When the structure failed, attention was given to improving and thickening the top or oiled part of the structure. Soon it was discovered that it was not this top layer of the whole road structure that was supporting the load, or, in most cases, not supporting the load, but that something underneath was needed. So, attention was given to the base course, and its design and construction was studied and a much improved base course technique was developed. Finally, it was found that there was still another important part of the structure, and the grade itself was given study with the science of soil stabilization entering into the picture. During the course of this development, we kept a fairly thick wearing course, a gravel or crushed rock course, even after the subgrade itself was improved by removing poorer soils, cross hauling, and compaction, to optimum density topping with A-1 soils, pit run gravel and other features of design. All of the improvements have increased the unit cost of the highway structure and it is evident from the papers prepared for this section that in order to capitalize on the progress made on the design of the structure, we must study it from the bottom up and not from the top down as we have done in the past. Engineers recognize now that with proper foundation construction the oiled part is needed for the wearing surface only, and the papers submitted show that great strides are being made in stretching out the taxpayer's dollar by the use of surface treatments inverted penetration treatment, blotter treatments, and single and double shot treatments, all of which are fundamentally the same in principle although the details differ in the various states and provinces. This type of surface is being used on the lighter traveled roads. True, it was discarded by some states in the past, but it is also true that the improved foundation design and technique warrants its being used on lighter traveled roads which constitute the bulk of the mileage on this continent.

Three of the papers prepared for this conference cover this type of construction. Both Mr. Tregillus of Alberta and Mr. Sedillo of New Mexico stress the importance of the foundation. In Alberta, on one project described, sheeps-foot rollers are being used in compacting the grade. The base course was designed after testing the samples of the subgrade material and the gravel to be added. The gravel was crusher run two inch minimum size. 1,500 cubic yards of this material were used with which was mixed sand or clay and some of the subgrade material. One-half of the required amounts of the various fractions was mixed spread, watered, and rolled, after which the second half was handled in the same manner. Cost, exclusive of gravel, was $400 per mile. In New Mexico, similar methods were used, either bringing in new gravel or utilizing the gravel in place on the road, removing oversize and adding binder, harrowing or processing, watering and rolling where necessary. The completed base course is tested with a straight edge templet and trued so that there is a variation of not over ¼″ in ten feet.

In Alberta, the finished base is primed with ¼ gallon per square yard of MC-1 or SC-2, MC giving better results.

MC-3 and 4 and SC-4 and 5 applied at the rate of ⅓ gallon per square yard were used on different sections as binder after the primer was absorbed and covered with ¾″ maximum size crushed gravel applied by means of spreader boxes, rolled with pneumatic rollers, and finished with a 6-ton three-wheel roller. After some time, a seal coat of MC-4 with ¾″ aggregate was applied. Mr. Tregillus points out in his onclusion that slightly damp surfaces absorb priming oil quicker than ry surfaces.

In New Mexico, the ¾″ aggregate is spread on the base course and he asphaltic material placed, after which another application of asphaltic aterial is made ,the surface being trued with a maintainer and rolled. his is the so-called ''single-shot'' method.

In the double-shot method, a prime of MC-1 is used. The asphalt seal s applied and covered with ½″ maximum size aggregate after which a econd application of asphalt is made and ¼″ aggregate applied. Mr. edillo tells us that results have been gratifying. On one experimental roject where both methods were used, it was impossible to distinguish etween the single and double shot treatment by casual inspection. Costs un from $1,400 to $3,000 per mile.

Mr. Collier of Oregon outlines for us their practice for this type of urface. They have 2,400 miles of oiled surfacing of ½″ to 1⅛″ in thickness, using crushed rock or crushed gravel with SC-6 or tar. A light ingle shot specification suggested is: Broom surface, spread SC-6 at 0.25 allon per square yard and cover with .006 cubic yard ¾″-½″ and .004 ubic yard of ½″ to ¼″ aggregate and roll.

The expirience in Idaho as given to us by Mr. D. M. Faires, Maintenance and Construction Engineer, shows that they have constructed surface treated roads at a cost of from five hundred to seven hundred dollars er mile, but that these treatments were often applied without any special reparation of the roadway and they went to a road mix mat 2½″ by 22 eet, costing $3,500 per mile. ''Thus, what began as a low cost, intermediate type of bituminous surface, is rapidly being advanced to the lass of heavy duty paving,'' Mr. Faires also emphasized the necessity f an unyielding foundation. Base courses are carried over the full width f the subgrade. Shoulders should be as nearly impervious as possible, and t is suggested that the oiled mat be carried over the full width of the oadway. This might create a traffic hazard because of the lack of contrast between the shoulder and the traveled way.

Diverting for the moment from Mr. Faires' paper, Wyoming has been studying this same problem. Their design for several years has called for base course over the full width of the grade. On main trunk roads, a 24 foot mat has been placed. So far, shoulder breaks have been eliminated. However, we are now constructing the mat 30 to 32 feet wide with no shoulders, as the cost of oiling the additional width is expected to be more than offset by saving in shoulder maintenance. It may be necessary to use three stripes on this work instead of one (in the center) as is now used.

In Idaho, MC-3 or SC-3 is being used in general. Stationary and traveling plants are used although best riding qualities are obtained by road mix methods. One project now under construction calls for the use of SC-6 and is in an area that has the extreme range of temperature of 160 degrees. ''It will be interesting to observe if the mat has sufficient elasticity to withstand the extremes of heat and cold.'' In this connection, the same question arises about bituminous construction in general.

With fills compacted to their maximum density, dense graded base courses thoroughly compacted, harder asphalts being used in the mats and in the seal coat, and these asphalts getting harder with age, are we not, by the improved design and technique, obtaining an unelastic structure which can not help but crack with the expansion and contraction caused by extreme temperature changes which take place over such a large area?

Mr. T. V. Fahnestock, Bituminous Engineer of North Carolina, outlines for us the development in the use of rapid curing cut backs in graded aggregate mixes. Slow and medium curing liquid asphalts did not prove satisfactory, largely because rain during the construction season caused difficulty in the mix. Parenthetically, it is to be noted that North and South Carolina have 46 inches of rain fall; Portland, Oregon, 41.6 inches; while Cheyenne, Wyoming, has 15; and Winnemucca, Nevada, has 8.5 inches. Further, similar differences apply as to relative humidity and, consequently, rate of evaporation. This point is often lost sight of in discussing the types of construction used in different areas.

Cut back asphalts allow wide limits in the grading of the aggregate; however, it is suggested that the minus 200 material should be limited to 10% or less unless mixing is done in some type of plant. Where local sand is available, a good surfacing can be constructed with 1 to 3 per cent less bitumen than required for hot mix sand or hot asphalt. Seal coats will be discussed later in this review, but it is to be noted here that no seal coat is required on this type of surface as it is practically impervious. Also, a first class non-skid sandpaper finish is obtained. It has been found that in these sand mixtures a moisture content in the sand up to 8% does not prevent mixing. A grey appearing mixture is produced under this condition. This does not affect the final result, as the moisture dries out and the particles of sand become coated. This does not apply, however, when the mix is composed of particles larger than No. 10. From laboratory tests and trial mixes, an extra rapid curing RC-3 has been set up. The new specification gives a material that will allow the mix to be spread and rolled immediately.

Mr. H. G. Groves, Bituminous Engineer of the Yale Oil Corporation, describes two city jobs in North Dakota. On the first project described, the base course consisted of gravel and soil binder mixed in a stationary plant, and after being laid to the proper section, was compacted to a density of 135 pounds per cubic foot with a pneumatic tired roller. MC-1 prime coat was applied to the finished base at the rate of ¼ gallon per square yard. Gravel, to which fine sand was added, then separated on a ¼″ sieve, dried and recombined in the plant, the moisture was held down below one per cent. Four per cent of SC-6 oil was added. A 2½″ thickness was used costing 50 cents a square yard and the 4½″ compacted base cost 31 cents per square yard.

On the other project at Ellendale, a 4″ base was used with a 3″ mat. The gravel and soil binder were mixed in a Barber Greene machine. MC-1 prime was applied. A Barber Greene machine was used for mixing. Before laying the mat, a fog coat of MC-1, about 1/12 of a gallon, was applied.

Street construction in Colorado Springs is discussed by Mr. C. H. Bryson, Superintendent of City Streets in Colorado Springs, Colorado. Dense graded mats are used exclusively on account of the character of the material which is available. The material is processed in place because of unavailability of plants. 50% of the areas of the city is covered with natural sand or gravel, thus requiring no base course. Some of the streets were on adobe clay. Only one inch of gravel was in place on some of this adobe foundation, but base failures were comparatively few. Adequate bases are needed (speaking now in general), but when adequate bases can not be afforded and property owners are demanding action, mats must be constructed and maintained on inadequate bases. Mill slimes were mixed with the local gravel to provide the 200 mesh material in the oil mix. Mr. Bryson notes that while MC-3 was used on several streets, SC-3 was used on practically all of them and the results were not noticeably different. Seals of RC-2 or RC-3 was placed 3 to 5 years after the mat was laid. Construction methods are similar to those used in other areas. It is to be noted that SC-3 prime is recommended for similar work. SC-3 is also used for mixing (using the same grade for mat and prime saves storage facilities). Pneumatic tired rollers are used for compacting the mat. A steel spring wire brush is attached to the moleboard of the finishing motor grader. Coarser aggregate is being used and the wire brush helps to fill the surface voids. Colorado Springs has an equivalent of 160 miles of 20 foot roadway. Streets were constructed to a thickness of 1″ although airport runways and suburban roads were constructed to 2½″ thickness. This is another example of necessity forcing low cost construction, and the engineer taking advantage of ideal soil and moisture conditions and using locally available material.

Mr. G. A. Draper, Materials Engineer of the Mississippi Highway Department, presents conclusions, based on experience, covering several points in regard to design of bituminous surfaces. High stability surface courses are not necessary now that adequate foundations are being constructed. The inference is that high stability subgrades and base courses carry the load and less attention is required to be given to the surface course.

The second point Mr. Draper calls to our attention is the imposition of strict requirements regarding penetration. The necessity of these strict requirements is questioned. He cites experiences on hot mix work where they experimented with low penetration working up in tank car lots to higher penetration, and states it is difficult to observe, under traffic, any difference between 30 to 50 points in softer grades, and 300 penetration did not seem to be too soft. These observations are confirmed by experiences in other states. Stability can be obtained with proper shaped aggregates rather than with a highly technical gradation of aggregate and ''almost any bitumen that will keep the water out'' and ''not separate them to such an extent that mechanical stability is broken up, will be satisfactory.''

Suggestions made for building satisfactory surfaces are as follows: Use agregate with highest inherent stability, rather than trying to obtain stability by the use of hard bitumen. High stability, aggregate permits the use of relative high percentages of high penetration asphalt. Again referring to Mr. Collier's paper, it is to be noted that Oregon's practice in their higher type surfaces depends largely on the interlocking qualities of the aggregate, having 1,100 miles of bituminous macadam on which 121 to 150 penetration asphalt is used.

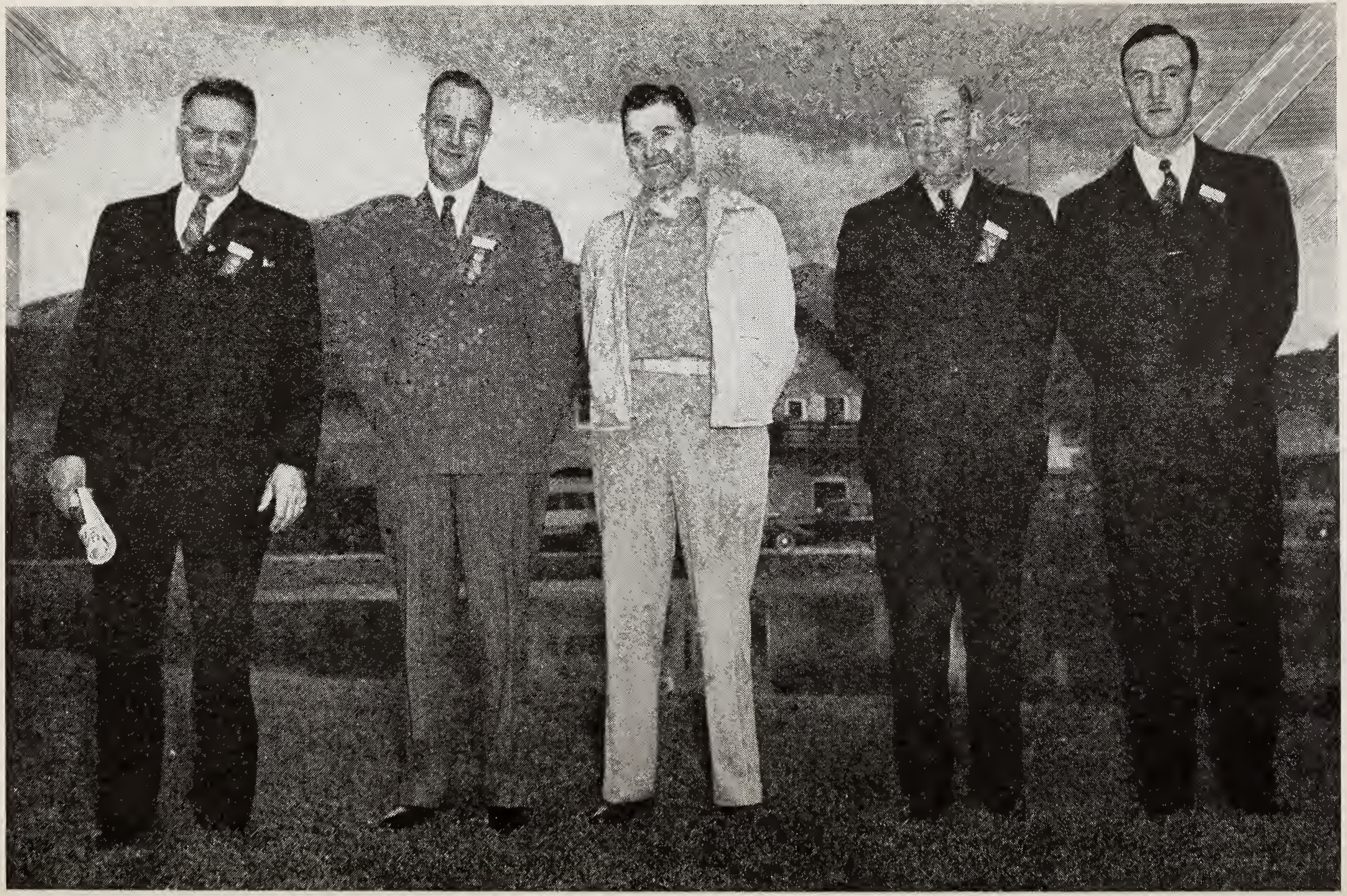
Mr. Draper emphasizes the point that high densities are not needed if the surface itself is dense and water tight.

Where high stability aggregate can not be obtained and it is necessary to use local "marble" or "ball bearing" aggregate, lower penetration asphalt must be used and greater care must be used in determining the percentage of bitumen.

Mr. Art Loforth, Construction Engineer of Nevada, gives us a paper on seal coats. It is to be noted that with the exception of the sand mixes of North Carolina, all of the papers we have before us stress the necessity for sealing. After experimenting with other grades, Nevada has adopted RC-2 grade for sealing. Seal coats are needed to bind the top portion of the mat to reduce ravelling, to liven the surface, and to waterproof it. Bleeding of sealed mats is not always caused by excess oil. It has been observed that when mats had been laid in the rain, even though proper precautions for drying had been taken, bleeding took place. It is further suggested that in the case of medium curing types, the solvent has not been sufficiently eliminated before laying.

When cover material is used with the seal, the amount of asphalt material varies from 1/6 to ¼ of a gallon: 12 to 15 pounds to the square yard of ⅜″ material is used. We must remember that in a properly constructed highway (subgrade, base course, and wearing course), the seal with or without chips is the only part that actually wears. Therefore, it is important that all angles of this feature be given careful study. When the temperature is too cold, the cover material kicks off. In applying the bitumen, missing sprays is another potent cause of failure of the chips to adhere. Experience caused Nevada to specify that bituminous material shall be spread not more than two minutes in advance of the cover material. However, this was found impracticable and it was found that by allowing 2,200 feet on an eleven foot width to be put down before applying the cover, no loss of screenings occurred.

Careful control, the use of first class equipment, especially mechanical spreaders, is emphasized by Mr. Loforth in his conclusions. In this day and age there is no excuse for the long, narrow streaks of ravelling surface due to missing sprays or sprays ejecting a deficient amount of oil. The responsibility of eliminating these streaks rests with the manufacturer, the operator, the contractor, and the engineer on the job.



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(Left to Right): C. F. Seifried, H. G. Groves, A. B. Nuss, Art Loforth, R. M. Schwegler. (Mr. Sedillo and Mr. Johnson not shown.)

Conference on Design and Construction of Bituminous Surfaces

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Cheyenne, Wyoming

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Mr. Seifried: Gentlemen, there may be considerable repetition, but don't think it hurts us to review the points that hav been brought out. In reading the summary of thes papers, I will skip anything which I find has bee thoroughly covered before. There is bound to be mor or less overlapping. We can't help that in a Conferenc of this kind but, as I mentioned to some of you, I though this morning we could perhaps make some effort to bridg the gap. We have this great mass of literature on technique, research and fundamentals, and still there are lot of the actual construction men, contractors, engineers the practical men if you will, who for some reaso or other seem to be unable to make use of a great dea of the information that has come out of the research tha has been carried on. I know the college professors her realize that one of their hardest problems is this matte of bridging the gap between knowledge and research, an building that bridge across so the ordinary, untutored human being can make use of it in his actual work. Using that bridge analogy, we could perhaps make it a two-span bridge. The first abutment will be the research man, and there is the gap that has to be bridged from the research man to the materials man. The bridge from the materials man to the actual construction man is the difficult one to span. This morning, I thought we could perhaps keep that general idea in mind.

The review of the points brought out in the various papers was comparatively short. I think it would be very interesting to go through some of these points and get the experience of some of the other states. Perhaps the technicians can answer some of our problems. Perhaps we can bridge that gap by their knowledge of the materials. Perhaps we can get some points that will be helpful to all the construction engineers, the practical men, if you will.

There are several questions that have been submitted and I hope some of us can bring out these points and perhaps take issue with some of these papers that have been submitted. There have been comparatively few controversial subjects brought out. Mr. Draper, of course, mentions some in his paper, which I commend all of you to read, as well as the others.

I wonder if we can perhaps start from the bottom and get some experience from the states that will be helpful to the others on construction considered as a whole. I know it has been taken care of in previous discussions, but just how far should we go in the matter of compacting the fill itself, starting right from the beginning? Mr. Muir questioned yesterday whether we are getting results with the new type of equipment, whether we are getting compaction. I don't know about that, but I do know that in Wyoming we have carried a running tabulation of the quantity of material in our cuts and our fills, just taking all of them. Of course, there are several million yards of material placed, but in 1933 the average shrinkage factor was zero on practically all the work that was done. There was some rock in it, but for the purposes of this tabulation there wasn't enough rock to affect the figures. Going on down through the years with the use of the heavier equipment, even before we had the present methods of soil compaction, that shrinkage factor has gradually gone up, so that the last tabulation we made showed a shrinkage of 10 per cent.

Without trying to take issue with Mr. Muir on that question, I thought it would be interesting to call attention to that tabulation. With these improved methods of construction, we are getting better compaction. In the last few years in Wyoming, it has increased from zero, on the average, up to 10 per cent. I wonder if anybody else has any figures or experiences on what they are doing, the results they are getting, and whether we are perhaps going too far on this matter of subgrade compaction.

Levi Muir:
Utah

We had a project, the one I told about that we did in the winter and didn't haul any water to the job. It was in flat country and rather a light job. We specified putting the sheep foot roller on the ridge of the surface before we went to work with the grading. We estimated that lowered the original surface possibly 0.2 foot. You can believe this or not, but that job showed a shrinkage of 67 per cent. We think a lot of it accounted for that lowering of the grade to begin with. That 67 per cent probably couldn't be taken as anything you would get very often on a project, because it was quite light and anybody knows that just a very little bit of difference in elevation on a light project will cause a very great change in the measured quantity.

Our engineering department is putting the materials department on the spot. They are demanding the estimated shrinkage in advance of construction. I think in some of the gravelly and sandy material we found only about 10 per cent. I don't know how close we will hit it, because I think it is pretty hard to tell. Anyway, they are demanding that.

I think we had another project that showed 25 per cent shrinkage, and that will probably be more nearly what we will get under average conditions in Utah where we are dealing principally with silty soils. Soils we can test by the regular Proctor method, we ca compact about 108 to 110 pounds to the cubic foot dry It will probably run about 25 per cent, and of cours where rock isn't involved.

Mr. Seifried: Have some others had some experience along that line

In regard to the different types of soil, sandy o gravelly soils, is it necessary to do any watering? Perhap a nominal amount of rolling is all that is necessary What I am trying to say is this: When we have obviousl sandy and gravelly soils, is it necessary to apply the same technique as we do when getting up into the clays, silts, and heavier soils? In other words, coming back to Mr. Draper's paper in general, haven't we something there that has a certain mechanical stability and perhaps we don't have to have it so dense? Can somebody help us out on that angle?

O. L. Stokstad:
Michigan

In Michigan we use the sheeps foot roller for compacting plastic soils usually with moisture control. In building an embankment at a bridge approach or across a gulch, we sometimes use pounding for consolidating sandy and gravelly materials.

Mr. Seifried: May we have some more experience on something along that line?

Some of you know, and perhaps some of you don't know that in 1752 a man by the name of Brindley was quite an engineer and made considerable progress, especially in the construction of canals, in England. I remember, in reading his life years ago, that when Mr. Brindley was called out on a job at various times some of these canal banks were leaking and they were having trouble with them. They didn't know how to construct them. Mr. Brindley had one cure. He would always say, "Puddle, puddle!" I have thought perhaps Mr. Brindley's methods were crude in mixing soil mortar into the fill and puddling it, but perhaps he was our first soils engineer, and perhaps we should give him credit for getting us started off on this.

I don't know whether it was brought out enough yesterday or not, but practically all the states brought out the point that they are compacting their fills from the grass roots up, you might say, carrying it right up in layers, and applying the same specification down at the bottom of the fill as they do at the top. I think it is generally agreed that that should be done, but in our state we have emphasized that in case there has to be any slacking off there will be absolutely none on the shoulders or on the top eighteen inches of the fills. In other words, in the way contractors operate, the weight of their equipment, their organization, and the operators, aren't we inclined a little to concentrate our compaction and perhaps our testing more in the center or perhaps the middle two-thirds of the fill, rather than get out on those shoulders and get that compacted more there? That is a practical construction problem. There is nothing technical about that. I just wonder if any of the others have any ideas on that, emphasizing compaction on the slopes and shoulders as they build up the fills. We all insist on building it up in layers, but I think the importance of putting it out on the shoulders and slopes should be stressed, as we bring it up. I wonder if anybody has thought about that or observed that. I would like to get some few remarks on that angle of it.

I am just trying to pick up a few of these points of yesterday. I don't think we should review them too far, but I think we should get down to the matter of actual operation on the ground.

One more point: Perhaps somebody can help us out in connection with these Western states where it never rains. Incidentally, when Professor Housel was telling about taking the excess moisture out yesterday, I just wondered if it wouldn't be seventh heaven for some of these construction engineers in the intermountain states if they ever got to a place where they had too much moisture. With these dry, fluffy soils and dust, it is a problem of getting water. In the Eastern states, we find they have to watch that they don't have too much water. As Professor Housel says, they sometimes leave more water in than the book of rules states.

Perhaps somebody has some ideas on this: We have been very much concerned about getting water, and we have thought some of letting our jobs in the fall, perhaps, going over the whole project and tearing up the cuts, and tearing up the borrow pits, leaving them lay there in loose condition during the winter and letting the winter moisture go in there; then go out in the spring as soon as we can and take advantage of the winter moisture. If we let our projects late in the fall rather than in the spring, we may solve our water problem in that manner. I wonder if anyone has any ideas on that, or any particular thought on it. Nature does give us some water in this country. Is it practical or possible for engineers to arrange their programs in these various states so as to perhaps conserve some of the winter moisture and use it in the spring?

V. A. Endersby: California

It is well established in experimenting with dry, granular soils, that vibration at a high frequency under a small load is much more effective than in compressing under any amount of pressure you can put on. So I was wondering whether anybody had experimented with any of the modern vibrating machines, and so forth, in trying to consolidate some of these light, dry soils in a dry region.

Mr. Seifried: Did you all get that? Mr. Endersby brings out the point that perhaps by vibrating these dry soils in the dry regions, agitating them, if you will, we would perhaps not use so much water. His point is that we all know when vibrated we can pack them down better than we can with a steady pressure. Has anybody had any experience to give any thought along those lines? It may be that is what we will have to come to. As a matter of fact, when you drive across these prairies you wonder where the water is and what they do. It seems that the contractors can find water by drilling wells, and in a great many of these places that look bad water can be found at comparatively small expense. On some of our jobs, we have had to haul water in. In at least one case we had to haul it in by rail. Those cases are few and far between.

I understand this is not a technical session, but is to get advice to help solve the problems in the field, problems that come up on the ground. I told some of the men last night that this was a foolish question session, and I hope you won't require me to ask all the foolish questions. I hope some of the other brothers will ask some of them, so we can bring out some of these points we are worried about.

Coming on up from the construction of the grade itself, I am wondering just how far the practice of cross hauling has gone, making use of the A-1 or the better soils. Just what progress has been made in building perhaps the top 18 inches out of imported soils, leaving the poorer soils not closer to the surface than perhaps 18 to 20 inches. I wonder if any of the states have gone that far and have specified that on all their work, where they could advantageously without too much cost. Has anybody had any experience in that? Can they help us out on that particular line, the matter of cross hauling, selecting the materials for fills themselves and doing away with the bad soils close to the surface?

Now we come to the next layer. A great many states, in the West especially, have gone to hauling in pit run material, fairly thick pit run gravel subbases. I don't think there is any particular point to be brought out on that, but the question does come up, as was mentioned in one or two of these papers, as to how thick those layers should be, how much rolling should be done, if any; how much refinement we should go into in the construction of these pit run bases in grading toward the top of the surface. Can anybody give us any results of their experience on that? I know those questions come up in the field, and perhaps we can get some help here in solving those.

If not, the next layer in our structure is the base course itself. A question comes up, which has been brought up before, and on which I would like to have some discussion. Just how much binder should be used, the effect of the binder, how much clay should be put in, and how densely graded we should make those base courses. Can someone help us out on that? I do want to refer back to the formulas for base thicknesses, but before that how about this matter of soil binder? Understand, I am not talking about chemical or other binders that are brought in, but the matter of soil binders.

W. S. Housel:
Michigan

Perhaps I might get some discussion started on that by putting forth an idea which I have always felt deserved very serious consideration, and that is that the most fundamental method of designing any mixture of that type is in going back to the old princples we used of proportioning materials by absolute volume. It seems to me it is a very serious mistake to overfill the voids in these base courses, or in any mixture. In the base courses you are talking about, you would be overfilling your voids if you put in too much clay. I believe the trend in base course construction in Michigan during the past two years has constantly been toward cutting down on minus 200 material, getting closer and closer to an ideal grading in which the clay does not overfill the voids. Then when moisture is absorbed by the clay and the clay swells there is not so much clay in the mixture that it begins to squeeze out on the surface or break up the aggregate interlock.

In the maintenance stabilization, that is, the work that is done by county maintenance crews, it seems to me—at least in driving over our state—that they have gone much too far in putting clay into stabilized gravel bases. It makes a beautiful billiard ball surface when it is dry, when the moisture condition is just right, but with a few rains and excess moisture the clay swells and begins to sqeeze out of the gravel and you get a surface coating that is very slippery and very hazardous. In addition to that, there is some decrease in stability of the aggregate itself due to the excessive volume change of the clay.

I have noticed in the discussions here in the last two or three days considerable emphasis on the P.I. of the mixture. My idea has always been that the plasticity index, which is a test for measuring the moisture-holding capacity of the clay, should be a quality test for the clay By selecting a high P.I. clay you may be able to introduce enough cohesion into the mat so that smaller quantities of the clay are sufficient. If you use a low P.I. clay you may have to put in too much clay. I wonder how many men, who are dealing with the design of mixtures for base construction, check the amount of clay from the standpoint of overfilling the voids. I should think that would be one of the most important considerations and the real basic method of designing the mixture.

Mr. Seifried: Professor Housel, don't you think we lose sight of the fact that we do put the mat on top to help distribute the load, and really the purpose of the clay binder is to hold that base course until we get a top over it to distribute the load? Aren't we neglecting to make use of the mat itself, the wearing surface itself?

We drive over these roads and say, "Isn't that a fine base?" Do we forget the fact that perhaps there are different mechanics set up after the mat is put on there? Even in this dry country here, from some place moisture does get in under there. That billiard ball surface is not a muddy surface when you put the mat on, but after perhaps the mat is placed it is a slimy, gooey base rather than a dry, billiard ball surface. What I am getting at, as I started to say in the beginning, is this: Can't we perhaps cut the clay down considerably, holding just enough in there to keep it stable, to keep it from raveling out until we get the mat on? Could you go into that a little?

W. S. Housel:
Michigan

I believe that is absolutely true. You do have a change in action when you put the impermeable surface on top, and due to cutting off the evaporation we so often see the moisture accumulating in the base under the bituminous surface, particularly on clay subgrades or where the drainage is questionable. So that there, too, not having too much clay is an important consideration and it seems there should be just as little clay as is necessary until you do put on the bituminous surface. I can't help but feel, in both of these cases that a trend toward smaller percentages of binder, just enough to allow the base to set up, is a step in the right direction.

C. E. Shumate:
Colorado

I would like to come to the defense of clay in one respect. We try—understand, try—to maintain a shoulder on bituminous roads in Colorado most of the way. A lot of our bases have been built without very much clay in them, a very low P.I. The water coming in off the shoulder, running off the mat into the shoulder, right through the base and into some of the subgrades which we have, has caused us an enormous amount of trouble and a lot of expense. If there is sufficient binder soil in that base material to hold it firm and make it more or less impervious to water until we get our bituminous mat on, we are not very much concerned with what is going to happen to it then. We have miles and miles of bases where the clay content is so low that they take water just like a sieve. Some of those bases are a foot or 18 inches in thickness, and the soil underneath them is of very low quality. I believe we have a little more rain in Colorado than they do in Wyoming, but not much, but from a week to ten days following a rainy spell we begin to have this failure due to the saturation of the subgrade underneath the base.

While I agree that the excess amount of clay in that base is detrimental, at the same time I don't want them to take it all away because I think it performs a very important function in that respect.

I. E. Russell: Wyoming

At breakfast this morning Mr. French of North Dakota brought out one point. Professor Housel said we shouldn't use too much clay, and I quite agree with him, but Mr. French stated that on one job they had in North Dakota they had the experience that, although the clay content of the mix was what they thought about right, the excessive rolling done on the job with rubber tired rollers seemed to create a suction and apparently robbed the clay from the base and brought it right to the surface.

Leon Gottlieb: Alabama

We have had lots of experience where the base course material contained more clay in the top two inches caused by continuous wetting and rolling operations. On our sand clay bases, we give the road a final machining or hard blading just before priming in order to machine off the excess clay on top. On our clay gravel bases we permit the base to dry out somewhat, and then machine the base again in order to remove the excess clay before placing the prime coat. If this phase is not guarded against, it is possible to lose the surface treatment because of the excess clay right under the prime coat. In our state we use close to 20 per cent clay in the material passing the No. 10 mesh. We don't have any trouble mixing, shaping and compacting the base course if experienced personnel controls the work. We are very careful in the final machining operation.

H. H. Houk: Washington, D. C.

There is an actual tendency for these finer particles of clay to work up to the top of the base course, which of course increases the clay content in the top of the base course.

W. S. Housel: Michigan

Mr. Chairman, right along the line of this discussion, I would like to read a few of the conclusions which Dr.

Winterkorn abstracted from the investigation by the Bureau of Public Roads which was presented Monday all of which I didn't read because the time was getting short. They are very much to the point in this discussion:

"The Bureau ran a series of tests on sand clay mixtures and sand clay gravel mixtures, two series of tests: Series 1, in which they used the same grading and varied the plasticity index by varying the amount of clay; Series 2 in which they held the plasticity index constant and varied the grading. Based upon this series of tests, specifications have been set up with recommended provisions to be used under bituminous surface treatment.

"Further conclusions offered from this investigation are:

"1. Control of grading, particularly by limiting the amount of material passing the No. 40 and the No. 200 sieve, is essential to insure satisfactory stability.

"2. Excessive amounts of dust, even if not producing a large plasticity index, should be avoided as such material, regardless of compaction, is conducive to failure under unfavorable moisture conditions.

"3. Thorough compaction must be obtained if trouble is to be avoided, and compaction obtained by vibrations is a closer index than that obtained by the Proctor method.

"4. Early difficulties in compacting mixtures possessing excessive grades and plasticity indices cannot be taken as indicating poor quality in service. Rather, the opposite seems to be the case. Compaction of base courses having plastic properties should be completed at maximum density before the surface course is applied."

I thought these conclusions were very much to the point in some of the discussions that have been given here, and I gathered from them that the series of tests which the Bureau ran brought out rather clearly the danger of having too much binder in the mixture. Even if you have difficulty in compacting your mixture due to smaller quantities of binder than you are accustomed to using, that isn't a sign that the surface is going to be bad. Their investigation indicates that rather the opposite is the case.

J. G. Schaub:
Michigan

Following Mr. Housel's remarks, we require the contractors to submit samples of clay at the same time they submit samples of their gravel aggregates in the manufacture of plant mix stabilized gravel. The purpose of that is to determine the amount of silt and sand in these clays, so that when they manufacture the gravel, the portion passing the No. 10 mesh and No. 40 mesh can be controlled and we do not overfill voids.

R. M. Schwegler: Oregon — We may be slightly unorthodox out on the West Coast, but we are very happy to say we can build roads without using clay, especially in a crushed rock. We had a lot of sad experience ten or twelve years ago where we used clay as binder in water-bound macadams. Our practice for the last five or six years has been to use no clay at all in our base courses, and in most of our base courses our P.I. is zero. By using water and rollers we have no difficulty in setting these materials up.

Mr. Seifried: Let's continue that just a little further, Mr. Schwegler. That is coming back to Mr. Draper's paper again. Aren't you depending practically altogether on the mechanical interlocking of the material? Don't you use crushed material more, and not the so-called ball bearing or marble type of aggregate?

R. M. Schwegler: Oregon — That is correct.

Mr. Seifried: I am afraid we sometimes get mixed up on this. In some places they have to use the ball-bearing type of aggregate. Unfortunately, a lot of us can't get hold of the crushed material, as Mr. Draper tells us in his paper. If we could have crushed rock entirely, I think perhaps a lot of this problem would be solved.

H. H. Houk: Washington, D. C. — I agree thoroughly that Oregon has some very fine roads, and that the practice in building roads is good in that state. However, the cost of their roads is entirely beyond the ability to pay of some of the other states.

H. E. Phelps: Washington — Bearing out Mr. Schwegler's remarks, the State Highway Department of Washington uses no clay at all. They do use a crusher dust. They are working with crushed rock. They put the roller on and compact it and work in a minimum amount of binder which, if possible, is crusher dust. I think the plasticity index is probably zero. That is particularly the case if they expect to treat the surface later. They have found, as Mr. Stackhouse brought out in his paper, that clay is detrimental there.

Mr. Seifried: Wouldn't it be more proper to say that crusher dust is filler and not binder? Do we sometimes get mixed up in the use of the terms "binder" and "filler"?

Leon Gottlieb: Alabama — When you mention building bases without clay, I am just wondering what the costs are. In our state we build top soil, sand clay or clay gravel base courses that cost us around thirty cents a cubic yard in place loose measurement including one mile haul. I am just wondering how such costs compare with bases constructed in other states where you don't use the clays and have to go through extra manipulations trying to get shape and compaction.

R. M. Schwegler: Oregon

We get crushed rock in Oregon and Washington from about $1 to $1.40 a cubic yard. We don't have natural sand clay or clay gravel materials. I may say the clays on the Pacific Coast aren't very good for building roads. They have P.I.'s from 30 to 40, and liquid limits from 50 to 80. I understand that in some of the Southern area you have clays with very low P.I. and very low liquid limit, so we are really talking about an entirely different material. The definition of clay is very poor. It doesn't tie you down to anything definite.

We don't have much gravel available in Oregon. We use it whenever we can, but it is not generally available locally. The state is mostly of volcanic origin and we have to use the natural rock available.

H. H. Houk: Washington, D. C.

I would like to ask the gentleman from Oregon if he has ever examined the float in their stream beds and investigated its properties.

R. M. Schwegler: Oregon

It all depends on the geology of the country. In Montana we have a lot of gravel with very good characteristics. The liquid limit is usually around 23, and the P.I. anywhere from zero to 5. On the east slope it is pretty bad. You get into more of the gumbo country. In Oregon there is very little gravel except in the stream beds, and in that case the material is usually coarse and poorly graded. The fines are usually a good concrete sand. By crushing we can produce fines which are satisfactory. Washington has more of a glacial area where they can use the gravels. They use them quite extensively in some areas, both as pit run materials and crusher run materials.

H. H. Houk: Washington D. C.

The point I was trying to make there was that we shouldn't conclude we do not have a satisfactory material for base course construction until we have thoroughly exhausted our means for prospecting the area in which we are working, because many, many times we have actually seen material shipped in to build and maintain a road for twenty years, and later found materials in the immediate vicinity of the project.

Mr. Seifried:

Professor Housel, I want to ask a question. Using a base course gravel, round, with the P.I. zero and no cohesiveness, if it were possible to get a perfectly graded mixture so that the fine sands and the finer material became filler, not binder; leaving out anything that has any cohesive qualities, could we get a graded mixture with no cohesiveness so it can solve these problems, with no P.I., but still have a dense graded mixture which will be stable?

W. S. Housel: Michigan

Theoretically, I would say it would be possible. I see no reason why it wouldn't be feasible, at least where you do have the material with the proper range of sizes, to produce a base of high stability so that when you get the bituminous surface down it would furnish the necessary confining influence to produce a stable system. The difficulties, of course, are in getting that base down and compacted so that you can get the surface on it. You have to have some binding medium to help out there, but you certainly can produce a mixture of very high stability, I should say very nearly as high stability without any clay in it as you could with the clay, because the aggregate interlock and the mechanical stability of those mixtures is the predominant factor as far as ability to distribute load is concerned. I wouldn't worry at all about a zero P.I. base course or a base course without any actual binder in it. It is a practical matter of getting it down and getting the surface laid. If you have enough cohesion to do that, I should think you could produce just as adequate a base without the clay as with it.

V. G. Gould: Iowa

In the Fundamentals Section yesterday, I believe I raised the point about the maximum limit for P.I.'s and left further discussion for the Round Table. It seems that we have reached that subject now, so I will discuss the matter more in detail at this time.

Professor Housel's remarks in reply to your question is, of course, the answer as far as the P.I. is concerned, but can you do those things and still make use of your local deposits without defeating your low cost aim? That is the reason I brought up the question of P.I.

We have one project now that happens to be an F.A. job, in which we have a natural clay binder in an area in which there is very little gravel available, except this particular source I am speaking of. It has a natural P.I. of about 12. The F.A. requirement, of course, is a zero to 6 P.I., and in order to reduce the P.I. to 6, sand was added, sand which was likewise available locally. I might say the project was completed late in the fall and the surface was primed to carry through the winter. This spring, when ready to place the surface course, it was found that the base was failing wherever excessive moisture was present. That same material had been used for years in its natural state, and had been found to be very satisfactory. What I had in mind about the P.I. was that if we had not attempted to reduce it from 12 to 6, we would have had a satisfactory base. At least everybody will agree it would have been in much better condition than it is at the present time. What actually happened was that with the earth shoulders permitting water to come in through the sides, or from underneath, the base material became like Cream of Wheat or oatmeal, resulting in complete failure. I am just mentioning this in connection with my remarks on P.I.'s the other day, and trying to tie in with the matter of making use of available material.

We have one other project of a natural pit run material in which only over-size was taken out, which had a P.I., I would guess, on the average of 3½ to 4 after the gradation had been secured. In that case our difficulty comes in in two phases: One of them is getting the material to stay in place until the surface has been placed; the other is that the increased loads under traffic are breaking the slight bond on the immediate surface of the base and causing some pushing or corrugating. Actually, the corrugations are in the top inch of the base.

Here is another point I have wondered about in connection with the maximum P.I. as determined by the Public Roads Administration with regard to the report on the various gradations of which you read the conclusions; that in securing their compaction the Public Roads Administration used a pneumatic tired wheel on a circular track with a uniform load, except gradually increased, in the determination of stability for both base and surface. In other words, they haven't determined the results from a heavy load on steel tires and many times in excess of that which they used in securing compaction.

I believe you will find there is a point in that. I just mention it for what it may be worth.

Leon Gottlieb: Alabama

We have had quite a bit of experience in compacting the clay gravel bases with P.I.'s of zero. We find it is necessary to use a lot of water during the mixing, shaping and compacting period in order to retain all the clay in the base course. Also, it is necessary to place the prime coat when the base is reasonably moist to prevent the sandy base from ravelling, it is also necessary to place the surface treatment mat as soon as possible. We have had to plow up many miles of sandy bases that have a zero P.I. If you know how to handle them—which is an operation requiring years of experience—you can get a satisfactory job using a very sandy base. Each construction operation has to follow close behind the preceding one on the base course, then the prime must be placed before the base gets dry. The surface treatment mat must be placed within a week's time in order to prevent traffic from ravelling out the sandy base.

There is one other point in connection with sandy materials, and that is the shape and size of the sand. We have run into conditions where most of the sand grains have been around No. 40 or No. 60 mesh, and of a round nature. Where we have such material it is very diffciult to hold the base in place unless there is a certain amount of cohesive clay present.

Mr. Seifried: That has been brought out previously in the discussion and in these papers. We are apt to get mixed up as to whether we are talking about round particles or angular particles, no matter whether we are talking about the big ones or the little, microscopic ones.

L. C. Campbell: New Mexico For many years we have tried to approach the optimum in binders of base courses, being compelled more or less to cut with the cloth and get materials that are available at reasonable cost. We therefore examine the materials which we have, and if it is found to be possessed of such characteristics as will do the job satisfactorily, well and good; if not, we determine the amount of addition of other material by taking samples of material of opposite characteristics which are feasibly available, examining them to find their characteristics, and then making a number of trial mixes. Then we determine the characteristics of those trial mixes and from that select the nearest approach to the optimum we can get with that kind of procedure which will be a firm support and yet sufficiently granular.

Mr. Seifried: I think this point Mr. Campbell is bringing out is very important.

L. C. Campbell: New Mexico Inasmuch as we have to more or less cut according to the cloth and do the best we can with the materials feasibly available, it has been our endeavor for many years, in an attempt to approach an optimum binder material, to take the material we had immediately available and examine it for its characteristics. If it was satisfactory to begin with, well and good, but if it was not satisfactory to begin with, then we took samples of other materials feasibly available and determined their characteristics. In selecting this material, we tried to select the material of opposite characteristics so that the combination would more nearly approach the product desired. Through that procedure we made trial mixes, varying the percentages of the various materials until the characteristics approached those at which we were driving. In other words, through that procedure we attempted to arrive at an optimum product.

W. S. Housel: Michigan

Mr. Chairman, Mr. Gould's experience and what Mr Campbell just said leads me to make several more remarks. It seems to me that possibly some of us are approaching the problem from two opposite viewpoints. By that I mean in one of Mr. Gould's examples the local material was evidently clay, or the clay was in excess, and their problem was to introduce as much granular material as was avaliable without making the cost excessive to improve that fine grained soil. Then we are also talking about cases where we have the granular material and are introducing clay merely as binder. We obviously have to approach these two cases in a somewhat different manner.

As Mr. Campbell pointed out, if you have a fine grained soil, you have to make a careful investigation of the characteristics, the volume change due to moisture content, and things of that nature, which are not so important if you have granular material to start with. The improvement of fine grained soils is right where soil stabilization, as developed during the last few years, comes into the picture.

It seems to me, then, we have to go beyond what has been referred to as physical stabilization and find out what additional measures we can take with these fine grained soils where proper granular materials to stabilize them physically are not available. That is the field that the manufacturers of cement, bituminous materials, asphaltic oils, asphaltic emulsions, and tars, have been studying in the past few years. Some of them are optimistic enough to believe they can stabilize almost any soil they encounter without having to haul in large quantities of granular material at considerable expense. The function of that type of soil stabilization, which you can call chemical stabilization, is to furnish a degree of improvement of the natural soil so that it will not deteriorate under excess moisture. The function of cement, for example, is to give you a permanent cemented structure. The function of the bitumen in general is to waterproof the structure. That is the real field of soil stabilization, and really the most troublesome problem we have is with the fine grained soils.

So in this discussion I would just like to point out that where you are building these bases, if you must use the fine grained materials, it is probably too expensive to haul in granular material or cut the P.I. down to the point where the soil will not deteriorate rapidly under natural moisture conditions. If that is your problem, while we may not have the answer now, the answer will be in soil stabilization.

That brings another factor into the problem. The possibility is to find out what can be done by the use of bituminous materials or cement in a range of low percentages to produce a durable soil structure from the natural soil.

. G. Gould:
owa

In making use of the local material on this first project I spoke of, where we added the sand to reduce the P.I., the resulting gradation was low in the amount of material passing the No. 40 screen. That is the reason the base became ''mush'' in the presence of excess moisture. In making use of the local materials to correct the gradation below the 40, we would again go back to the addition of soil, which we are now doing, resulting in a higher P.I. Thus we travel in a circle. We know by making use of bituminous materials or the cement as a stabilizer, which we have already tried in our laboratory, we can expect satisfactory results. However, their use will run the cost far above the use of local materials; that is a correction of the gradation through the addition of soil. The use of bitumens or cement is one of economics and is the reason it hasn't been done.

W. S. Housel:
Michigan

That is entirely an economic consideration, whether you can improve the natural soil by the use of admixtures to such an extent, and within a reasonable cost, so that it doesn't cost more to do it that way than to haul in the other material.

In the first example you gave, Mr. Gould, there is one point that is important. I understood you to say you sealed the surface and primed it, and failure resulted, even though these same materials were perfectly satisfactory in other places. By other places, I presume you were referring to gravel roads.

V. G. Gould:
Iowa

That is right.

W. S. Housel:
Michigan

I think that is an illustration you have all probably seen time and again. I believe there is an important consideration there. Putting on the somewhat impermeable surface which eliminates the evaporation from the surface and allow the capillary moisture to accumulate is probably the real answer to your difficulty in that project, with materials that were not giving difficulty in the ordinary gravel road.

Leon Gottlieb:
Alabama

I would like to ask Mr. Campbell how they mix the materials where they have to be changed from the natural state in order to get a material meeting the specification requirements.

L. C. Campbell: New Mexico

After having examined the materials available for the various characteristics, if we are not familiar with them to begin with, we just have to start mixing in the most natural proportion we can estimate to give us the desired results. Sometimes we might hit close to that, and sometimes far apart. Sometimes we have to make quite a number of mixtures before we get on the right track and approach the right proportion. After we have our characteristics, particularly as we combine the various materials which are available, we can more closely approach the optimum proportions as 1:3 or 1:4. After we get the materials, we can more quickly secure the approximate optimum combination by picking out, as a starter, what past experience with similar materials says appears to be the optimum we can get in the material.

Leon Gottlieb: Alabama

What I am asking is, how do you do your mixing on the road?

L. C. Campbell: New Mexico

After we have issued our findings to the field, giving them the proportion we have found in the laboratory or gleaned from our experiments in the laboratory to give an optimum product, then mixing in the field is a matter of construction detail which may vary widely according to the equipment they have available and in use. If the material is available locally, as frequently is the case, then the contractor, under the supervision of the inspector, may delegate a certain proportion of his force to the bringing in of one material and delegate another proportion of his force to bringing in the other. So far as the detail of mixing in the field is concerned, that may vary widely according to the equipment and conditions. After correct proportioning is cared for any and all preliminary mixing the material may have received as a result of handling, screening, etc., is supplemented with final blading on the road.

In most cases mixing starts at the screening plant: Units of the different materials are alternately dumped through the trap onto the bucket line; considerable mixing results in the flow of material through the screening plant, into the bins and then into the trucks. This mixing and/or any other it may receive in this and/or other handling is supplemented with final blade mixing on the road. Sometimes corrective material is dumped on the road with spreader dump trucks (on the material to be corrected which had already been spread) bladed back and forth to further smooth and better assure uniform application and then mixed with the blade, turning the windrow over from side to side until material by visual inspection seems thoroughly mixed. No mixing operation similar to concrete mixer, pug mill or other mixing plant is used.

. E. Russell:
Wyoming

I would like to have you tell us by what tests you determine when you have reached your optimum condition.

L. C. Campbell:
New Mexico

Before these various tests that are now in use were in general use, we had to do that more or less by main strength. In the old days we had what we called the field moisture equivalent. We had a tenacity test and a linear shrinkage test. More or less by trial and error and growth with experience, we finally determined what at least seemed to give us optimum results. With respect to the tenacity which we used before this came in, a tenacity of about 100 seemed to be as good as any we could determine for that purpose, and a linear shrinkage of not to exceed 4 per cent, and preferably less than that, and field moisture equivalent of less than 20. Of course we had some soils which were out of the ordinary, one of which was "caliche," more or less a law unto itself. While these tests are in terms of the old language, they are used to give an idea of what limits we determined. We have not thoroughly and completely correlated the old language with the new as spoken now, but we are making steps in that direction.

H. H. Houk:
Washington, D. C.

I think we should draw out the details of this mixing a little clearer. In the early days of base course construction we had failures even though satisfactory material was used because of improper mixing, and the importance of thorough mixing of natural occurring materials used in the construction and design of these soil base courses cannot be over-stressed. The use of proper equipment under proper conditions is absolutely necessary to obtain satisfactory results. You may have what theoretically is a perfect mixture, and yet have failures in the base course due to improper mixing, or non-uniform mixing. I think that was the point Mr. Gottlieb was trying to bring out when he asked Mr. Campbell what his process was.

Bruce M. Ross:
Illinois

While on the subject of mixing, I think it would be very interesting if we could hear from Professor Housel again on the extensiveness of plant mixing in Michigan.

W. S. Housel:
Michigan

I think I will refer that question to Mr. Schaub. He is the construction engineer and I think he can speak much more intelligently on this matter.

J. G. Schaub:
Michigan

Since we have gone into carefully graded gravel base aggregate, with the addition of soils and chemicals, we have adopted entirely the plant mix construction. We use the Pioneer type of stabilizer or other plants of the same type which have a clay disintegrator and a pug mill mixer. We get a very uniform mixture, and feel it pays us. At first the cost was a little high until the contractors became equipped. Now our costs average between $1 and $1.50 a cubic yard, in place. The cost of the gravel plant mix stabilization with clay, hauling and laying compares very favorably with what it formerly cost us to place and stabilize the gravel base by road mix methods We feel the more accurate and controllable results by the plant mix methods are worth a little more anyway

Mr. Seifried: I think the discussion is very healthy, and I think Mr. Gottlieb got what he was after on the points he brought up.

I don't know whether I am speaking in general or not, but I know in Wyoming we have used rather crude methods in mixing the binders, clays or sands, whichever we have to have, but in the laboratory we have endeavored to use rather accurate methods.

The point I am getting at is this same bridge. If we are going to get down to technical points in the laboratory, we can't go out in the field and mix these by hit-and-miss methods. We must keep our construction practice up with the laboratory technique.

T. R. Perry:
Iowa

This has been a splendid discussion of the top part of the road, but I am not quite satisfied to just pass over without any remarks or any discussion of the question which the Chairman brought up in regard to the top 12 or 18 inches of the road just below the stabilized or crushed rock base.

In Iowa this spring we had the worst breakup we have had in ten years, and, as a result, had a chance to observe what soil substitution really does on cuts and fills. One of the roads, which I think of now, built about eight or nine years ago without any particular attention to soil substitution, broke up both in the cuts and in the fills. During recent years we have used soil substitution and the location, type, and character of these treatments have been determined from surveys made before the grades were reconstructed, prior to the placing of the stabilized base. These soil substitutions stood out like a sore thumb this spring. They didn't click 100%. A few of them failed. Some were a little short on one end and some were a little short on both ends.

Nevertheless, it stood out as a very clear picture, and we feel justified in carefully studying the grades and soil conditions in the top 12 or 18 inches before we go to the expense of putting on our stabilized base and bituminous surface.

Mr. Seifried: Thank you very much, Mr. Perry. I think that is very important, especially in the last remark Mr. Perry made.

Let's study the subgrade and base course. Perhaps the laboratory technicians can't tell us exactly what to put in, but we can combine a little practical experience with the theory before we put expensive, thick mats on these questionable bases. Let's properly place our selected soil, imported soil, cross hauled material, base course, and then perhaps, as I remarked, put on a thin top course to waterproof it instead of a thick, expensive surface.

Mr. Perry brought up a very important point. I wonder if anybody else has anything he can give us on that. Fortunately, in some western states the places where there are bad soil conditions are confined mostly to the mountainous areas in comparatively small parts of the state, and that trouble isn't as widespread as perhaps it is in the eastern and southern states. I think that is all the more reason, especially in this Inter-Mountain and Western Rocky Mountain area where these bad soils are in the subgrade, we are justified in going to considerable expense in removing them and bringing in proper materials for this top 18 inches.

L. C. Campbell: New Mexico

With reference to soil substitution, I believe it is pretty generally conceded that the soil which is classified as A-1 is about the best material or the nearest approach to the ideal material that can be obtained. Of course, that may not always be existent within feasible distances, but where it is not, the next best thing that can be done is to take those materials at hand and, by experimentation, by combination in the laboratory, produce what you might call a synthetic material nearest approaching that classification. If you can approach that very closely, it seems you may have the problem solved as near as it can be solved within the financial limits.

W. S. Housel: Michigan

I would like to ask Mr. Stokstad to outline the procedure he is following in the soil surveys, and the use of subbases and sand and gravel, that is being done in our state.

O. L. Stokstad: Michigan

The first step in the study of foundations for highways consists of making a soil inventory; in other words, classifying the soils. This classification is the foundation upon which all following soil investigation and recommendations are based and serves as a means for becoming acquainted with the soil problems presented by a particular project.

Methods of soil classification seem to vary in different regions depending on the background of personnel and variations in foundation problems. It seems important that each state should develop its tools of classification so as to work efficiently without departing too far from some generally accepted standard. For the purpose of gathering general soil information Michigan finds the pedological system of classification very useful. If the entire highway program is to profit by soil survey information it is important that classification tools permit speed. Two to eight miles per day of soil survey is not unreasonable for one man, depending on the detail to be shown.

To make a soil classification generally useful to highway engineers Michigan has made effective use of charts tabulating design and construction information as pertaining to individual soils.

Most highway foundations, especially in glaciated regions, are composed of a wide variety of soils and it is seldom therefore that a project requires uniform foundation treatment throughout its entire length. From the soil classification survey we know where the sands, gravels, clays, silts, peats, etc., are located and it is a relatively simple matter to work out a system of cross haul so as to use these materials all to their best advantage. General practice now requires that all soils whose characteristics are dominated by their clay content be covered with a 12 inch sand or gravel subbase as a foundation for a surfacing.

The soil classification survey is used directly by the designers in preparing plans with very little laboratory testing necessary. The soil engineering highway technique as developed in Michigan therefore is principally a field activity. It is not expensive, not highly technical and permits a small organization to cover a lot of ground.

Mr. Seifried: I don't think we can overstress the importance of soil surveys. Also, we cannot stress too much the importance of making these soil surveys correctly.

They are working on a project in Wyoming now about which there was much concern. They got the soil samples and sent them to the laboratory and, using charts and diagrams to determine the thickness of the base course, the laboratory gave us recommendations that I was afraid were going to break the Highway Department financially.

The Construction Engineer was sent out and told, ''Look that over. See where they dug those holes and see if we can't do something to overcome that expense. We want to do it scientifically, and we don't want to get in wrong.'' He came back and told us that perhaps the man who took the soil samples at first was a little at fault, and perhaps the laboratory was all right.

The project was in what we call a fairly rough, rolling country, with considerable rock in it. Those fellows had gone out and dug samples two or three inches deep in the ground, and took no account of the rock from which most of the fills were to be made. They were having us place a heavy base course on this rock fill on the basis of samples taken on the surface of the ground.

I shouldn't tell that. That isn't the way we do all our work, but that did happen.

Are there any further remarks? Perhaps we have gotten through the base course toward the top, and we might bring up a few points on the wearing course.

I think we are all well agreed; in fact, it is becoming axiomatic—(I think perhaps at our next convocation it might be well to bar certain expressions, like ''adequate base courses'' and ''good foundations'')—that any wearing surface will hold up the road if the base and foundation are properly constructed. I think perhaps the committee had better send out a letter and say that everybody knows that, so we won't have to repeat that next time and can start from that point. We have learned that now, and I don't think it is necessary to repeat it any further. The main point we want to study and discuss is how to get them. Before leaving the matter of base courses, let's emphasize that. In our discussions let's get on the side of the rounded particles and discuss those. What you need is a certain amount of cohesion, density, and good grading. That applies not only to the coarse particles, but to the fine particles. On the other hand, the other school of thought, or the other area of the country, is talking about and using angular particles, crushed particles, if you will, for the stabilization. Let's keep those two separate in our discussions.

I mentioned in the beginning surface treatment. As we all know, there have to be a lot of miles of roads built, and they have to be built very economically. I mentioned yesterday that 83 per cent of the roads carry less than 15 per cent of the traffic, or something like that. I forget the figures, but these 83 per cent of roads have to have something done to them. It was mentioned in several of these papers that they have developed this thin treatment for traffic of less than—for a great many cases in our state, and I know in these other states—100 cars a day. Still we have to get some kind of oil surface on them. One or two points or questions brought up have been discussed before at these meetings, and perhaps we can touch on them briefly.

We need to keep in mind the amount of moisture. Mr. Gottlieb mentioned that, and we have discussed the subject back and forth. I am talking about surface treatment now. In order to be definite, let's say we are putting on a surface treatment on some kind of material, using an MC-1 to penetrate it, prime coat, if you will; coming along later with probably an RC-4—that is what it would be in Wyoming, and in a good many of these other states, as you noticed from these papers—then placing chips on that or, if we can't get chips, getting back to the other side putting on buckshot gravel or fine, clean, screened gravel. That type of construction we all know can be built for anywhere from $800 or $900 up to perhaps $1,700 a mile. We have something there that will carry the light traffic. With an inexpensive surface like that we are justified in spending something on the maintenance of it.

Coming back to my specific question, perhaps somebody can review what we should do about this moisture in the surface to be primed before placing the prime coat. Some advocate that it should be fairly moist in order to pull the prime coat down. I am talking to the laymen now, and I hope I will be corrected if I make any mis-statements. If you get too much moisture in and the temperature gets warmer, and the moisture tends to come up and flush the surface and cause bleeding, and of course if it happens later it forces the prime coat up, shoves up the RC, the sticking coat or whatever you want to call it, and causes the slick spots which are a potent cause for a great many automobile deaths. I wonder if any of you could tell us some experiences you have had in the matter of moisture, how much moisture we should have, if any, or whether it makes any difference.

Mr. Gottlieb, can you review one or two of your remarks on that?

Leon Gottlieb: Alabama

We use mostly tar for our prime. We have used some MC-1. If the surface is primed when the base is too dry, there is a tendency for the prime not to absorb into the base on account of the dust particles. It just lays on top. Therefore, it is better to dampen the road a little before applying the prime coat in order to give the prime a better chance to penetrate through the dust.

On the other hand, if the base is too wet, the prime will just lay on top and won't absorb at all. It probably won't ever be absorbed into the base and when such condition occurs bleeding may result. Sometimes the base is so wet as to cause the prime to peel off. In such cases, the prime is just laying right on top and may occasionally cause the mat to shove a little. That has been our experience with prime. We haven't made any tests, though, to determine the moisture content.

Mr. Seifried: It is mostly a matter of judgment, I take it, and experience, Mr. Gottlieb.

Does anybody else have any experience on that? I think it is an important point.

R. Flint: daho

I have had occasion to do some of that work in the field and, as Mr. Gottlieb said, any pockets of dust naturally do not absorb the prime, and immediately under traffic these places kick out.

I can't say how much water we used, but we did flush a short time ahead of the application of the prime coat and watched it carefully. When our surface showed a dryness and yet there was no water standing there, we applied our prime which, in the case I was on, happened to be SC-1A. We were rather liberal with the water quantity, just so it didn't show on top of the surface. We seemed to have pretty good luck, and experienced no bleeding afterward .

In one case, the profile and cross section wasn't just as we wanted it, and we applied enough water to that so that by means of blades and floats we could work that surface. We followed that with rolling, and then made our application, and it was very successful.

W. S. Housel: Michigan

I haven't any experience to relate, but this subject was discussed at the Biloxi Conference last year, and it seems to me it is rather important that we understand the mechanism that may be involved. Mr. Powers related experiences which checked with what Mr. Gottlieb and Mr. Flint have said.

I think the mechanism involved there can be illustrated from our experience with the preparation of laboratory samples of some of the stabilized mixtures. While you use the moisture to disperse the bitumen in the mixture and mix it, we find that a drying period in which that moisture is evaporated out is very important. I suggested that the dispersed bitumen is carried into the small throats of the capillary openings and seals the soil, when deposited there as moisture evaporates.

Translating that into terms of the phenomena you are talking about now, it appears that if the surface has been subjected to a considerable drying period and you prime it, you can't get any penetration due to the dust particles, and there is nothing to suck the bitumen into the soil structure. If you put an application of water on the surface, the capillary absorption of the dry base pulls that water in, and if the bitumen is put on at the right time it is pulled in with it. It follows, in the same way, that if you attempt to prime a base that is already fairly well saturated, you won't get penetration of the bitumen. When the weather gets warm again, you will have the bleeding that has been mentioned several times.

In connection with that bleeding, there are one or two points about which one might speculate. We have observed bleeding on some projects that have been stabilized with bitumen, in which we were absolutely certain the voids in the mixture were not filled with water and bitumen. When this is the case, it is difficult to account for the bleeding as additional compaction under traffic because the voids are not full. Yet you have the bleeding on the surface in warm weather.

It appeared to some of us, in discussing that problem, that probably one of the things that happens is that when the temperature gets higher and excess moisture is in the mixture, it vaporizes and may develop considerable pressure. This water vapor under pressure naturally tends to escape and may bubble through the seal coat, and perhaps in places actually lift the seal coat. If this conception is at all accurate, it brings out again the importance of not sealing these mixtures when there is too much moisture in them.

L. C. Campbell: New Mexico

In attempting to describe conditions rather than to absolutely fix values, I believe you will find by and large that the optimum, or close to the optimum, percentage of moisture in the aggregate at the time the bitumen is applied is that which will just barely cover the surface, whatever percentage that may work out to be.

Mr. Seifried: Have we anything further on that matter? Following through seal coats and coming to seal coats with cover, I wonder if we can get a little help in the problem we have. This is a practical field construction problem. What we are after is to get a certain amount—enough and not too much—of asphalt to hold sand or rock particles on the surface to form this sandpaper finish of which we have spoken. It is a fact that if we had perfectly ideal conditions, we could put a comparatively thin film of bitumen on the surface of the road, and perhaps put very small particles down, to an eighth or even smaller. That would be an ideal condition and you would get a nice sandpaper finish, but from a practical standpoint it is necessary to use larger particles. We can't manipulate those finer particles; we can't put them on in the proper way. I wonder if anybody has any definite ideas.

You noticed in these papers they use anywhere from one-quarter up to three-quarters. It seems to me all that is necessary is to get that surface covered, and if we can properly stick a quarter-inch or half-inch material that is what we should use. As mentioned in Mr. Moyer's discussions in papers and publications, the bigger particles form small bumps in the road, and when the bumps in the road are as slick as the surface we don't get the effect we want. I should like to know just about what size we should use, and the type of asphalts. The rapid curing cutbacks seem to be the popular ones, as brought out in our other discussions.

Mr. Loforth, have you anything further you can give us on that?

rt Loforth: Nevada specifies a three-eighths inch screen for a light surface treatment, with allowance of 5 per cent for oversize passing a one-half inch screen. Personally, that 5 per cent always strikes me as being a little too much in the way of oversize.

evada

California uses three-eighths. From observations in California, that particular size seems to be the most desirable. The larger screens, as brought out by Prof. Moyer the other day, cause bumps, and you get away from the sandpaper finish which is very desirable.

r. Seifried: Is there anything further on that?

I know we have all had the experience of using too small chips. We haven't perfected our methods of applying the chip coats. It has been brought up that the rubber tires on the spreaders are responsible for some of those pick-ups, rather than defective spray bars. I wonder if anybody can give us any help on that. This is really a mechanical problem.

Leon Gottlieb: We have tried the inverted penetration method of seal coating in lots of different applications. The difficulty we had with thin applications was that the surface to be sealed may vary in its absorptive qualities, whether it be a prime base or on older treatment or something else. Therefore, the open textured areas would absorb most of the sealing bitumen thereby not leaving enough on the surface to hold the aggregate.

Alabama

We have had some roads where we could put down .15 gallon of cutback and it would stick three-eighths to No. 4 mesh chips. On other roads where we put down .15 gallon of the same grade of material, it wouldn't stick any chips at all because all the bitumen was absorbed into the underlying course. For that reason we have gone into a little heavier treatment, using .3 and .35 gallon of bituminous material and using a larger sized aggregate. It gives you a little more safety, due to the varying conditions of absorption and surface texture, than you would have with the lighter mix, and also permits the use of a larger sized aggregate which produces a thicker surface.

Mr. Seifried: Your thought, Mr. Gottlieb, is that we have to go into the larger sized chips in order to overcome the varying conditions of weather, types of asphalt and types of material.

The time is getting short. I think we have done fairly well. I was a little skeptical and worried when I started and I certainly appreciate the fine discussion and help everyone has given.

I didn't intend to ignore the group of distinguished gentlemen sitting on the platform. I should like to give perhaps two minutes to each one of them to summarize what has been said this morning or, if they can't do that, perhaps they can give us some ideas.

Mr. Keeling, can you give us a couple of minutes and perhaps hit some of the high points?

R. C. Keeling: Kansas — I am just wondering if all the states are constructing their bases the full width of the roadbed, even though it is a crushed stone base. Do you in Washington?

R. M. Schwegler: Oregon — Yes, we do.

R. C. Keeling: Kansas — I think all of the different things taken up are about in line with what Kansas has been doing. On the seal course, we are using .3 to .35.

Mr. Seifried: When Mr. Keeling was speaking, it occurred to me what wonderful help we have here. They say we don't make progress in these things. Certainly we are making progress when we bring out our experiences and ideas. Mr. Keeling says Kansas fits in line. We all fit in line. We discuss these things, and rather than one state going ahead single-handed and making progress, and the rest of us not doing anything these other states do, the other states bring their experiences to us, and we are getting a certain amount of uniformity throughout the country. Practice and technique are going ahead.

Mr. Schwegler has been very kind and helpful to us. Have you anything further you could give us, Mr. Schwegler?

R. M. Schwegler: Oregon — I might attempt to describe our practice in forest and park work, as carried on by the Public Roads Administration on the West Coast.

We are attempting to stabilize the top 12 to 18 inches of our subgrade before constructing our base courses. We accomplish this whenever possible by cross hauling selected materials, presumably A-1, 2, or 3 group materials. We sometimes use selected borrow. Wherever it is possible to cross haul at very low cost and get a stabilized subgrade, we think that is the solution to our problem.

We are also more or less trying to establish uniform stability in our subgrade, which permits us to use a more or less uniform thickness of base and top course. We generally follow the practice of using what we call a double prime coat, consisting of .6 gallon applied in two applications, of an MC-2 or similar material on our surfaced roadway. This treatment usually carries traffic for a period of two to four years; in some cases a little longer. By that time, we consider our foundation has been proven. Any failure which is apt to occur has occurred in that time, and we are then in position to continue with a more permanent type of construction for our wearing surface.

We also follow the practice of priming full width in District 1. We prime from shoulder to shoulder. I think the small additional cost is very definitely worthwhile. It reduces the maintenance on shoulders and waterproofs the entire road surface. It protects our subgrade from any excessive amount of moisture. I might add that we found it desirable to avoid excessive brooming before applying our surface treatment. Our surfacing materials are usually fairly loose, and we follow the process of tight blading and rolling to prepare our base for treatment. Quite often we precede application of asphalt with a light application of water. As soon as the surface is dried to a moist condition and there is no free water apparent, we apply our asphalt and get very good coverage and very good penetration.

Mr. Seifried: Thank you, Mr. Schwegler. The question of full width was brought out in Mr. Faires' paper. I say "full width," which means lots of things: Putting on the base course the full width, priming it full width before putting on the top course or chip treatment, or whatever you are going to put on.

Mr. Johnston, I wonder if you could help us any further on that.

J. M. Johnston:
Idaho

I would like to make a summation on this business of the use of mineral binder. If you have enough money to give the job some kind of bituminous treatment don't put in binder; if you have not the money, put in the binder.

We know from experience that vibration is the thing that starts your road to coming up and going to pieces. If you can stop that with the use of clay binder, that is all there is to it; put in the binder. If you haven't the money to put on the oil treatment, that is about all there is to it.

Mr. Seifried: Thank you, Mr. Johnston.

Mr. Nuss, I wonder if you have any remarks you could give us on those things about which we have been talking. Don't try to cover all the points, but as many as you can.

A. B. Nuss: Nebraska

I had one little point on which I wanted to hear from someone in the audience. Before I ask that, I would like to say that practical experience, from the standpoint of the contractor, in the matter of moisture in the subgrade, shows that the most ideal situation you could have is an absolutely dry subgrade, and then apply water. That speeds up the contractor's operation in the field above anything else. Generally, you can get the penetration in a couple of hours' time. Otherwise, you would have to wait a day or two. It is a big help to the contractors. Of course, in the end that is reflected in their bid price if they can speed their operations.

Going on with this question of moisture, we didn't discuss anything about the moisture in the aggregate at the time of mixing out a mat, or at the time you are applying your oil to the aggregate and the mat. That is another important item in my opinion.

Aggregates that are dry and dusty don't seem to take a coating of asphalt very well, and require a lot more mixing. In some states specifications are such that you can't have any moisture in the aggregate at the time of applying the oil and mixing. As a whole, I wonder what states are doing that now. From the contractor's standpoint, the mixing operation is speeded up if there is some moisture in the aggregate. It seems to give a more uniform coating of asphalt to the aggregate. I was wondering what the practice is in other states.

Mr. Seifried:

Are there any comments? The point Mr. Nuss brings up is the moisture in the aggregate. I think that was brought out in some of these papers. I don't know whether I caught it in the review or not.

We have been so scared of having any moisture in our aggregate before applying the bitumen, before starting the mixing operation, that we may have gone too far. Mr. Nuss brings out that there is an advantage in having a little moisture in the aggregate before the application of the oil. Has anybody any comment on that, briefly?

T. R. Perry: Iowa

It might be interesting, and might even be shocking to you to know the per cent of moisture we permit in the aggregate in an inverted penetration type of construction. A number of years ago we limited the maximum amount of moisture to 2 per cent in the aggregate at the time of application. The contractors had a lot of difficulty bringing shipped-in material, which happened to get caught in a rain, down to the 2 per cent limit before applying, especially if the aggregate contained a little silt.

We carried on some experiments in the field using various amounts of moisture and we found that after the aggregate had been in the road for a year or two the amount of moisture automatically settled at about 2 per cent. We also found from these experiments that we could increase the amount of moisture up to 4 per cent at the time of application and still secure the same results. The result was that construction was simplified and speeded up very materially.

I do not want you to gather the impression that we actually incorporate 4 per cent of moisture in the mat, because that is not the case since a considerable amount of moisture evaporates before the aggregate is actually imbedded in the bitumen. I am satisfied that the per cent of moisture actually incorporated in the mat does not exceed the 2 per cent.

Mr. Seifried: Is there anything further on that? Perhaps in a practical way—I don't know anything about it—we can meet this hydrophilic and hydrophobic and megapoise business by putting in a little water and not worrying about it any further.

Is there any further discussion on that subject? I think that is very important and something we should think about.

L. C. Campbell:
New Mexico

The New Mexico limit is 2 per cent maximum.

V. A. Endersby:
California

I would like to point out that if you put it on a scientific basis, it ought to be considered in amount of surface area rather than percentage by weight. Those who are familiar with Hveem's oil line method of proportioning oil can see the analogy there.

R. C. Keeling:
Kansas

After the oil is applied to the aggregate, when there is moisture present, do you figure that the water can be gotten out of the aggregate by mixing? A lot of them do quite a bit of road mixing. I know we have gone higher than 2 per cent, and by considerable mixing after the oil was applied we didn't have any serious results. I just wondered about that. That is in mat construction.

V. A. Endersby:
California

All these mixtures appear to be porous enough so that they will lose their water, unless tightly sealed, and will reduce the water to the amount which is very tightly held by the aggregate. That bears out the experience which has just been mentioned, that it reaches a certain minimum and stays there. If you have too much water in the mix you get stripping.

Levi Muir:
Utah

In Utah, we have always considered the amount of water very important. We have two general classes of aggregate that we oil, one being sedimentary. In that case, in ordinary weather, if the materials are dried out reasonably well before the oil is added, they will mix out so that the percentage of moisture is one-half of one per cent or six-tenths of one per cent. We specify one per cent for sedimentary material.

Then we have another class of material which is mostly aqueous in origin. Any way we mix it, we can't mix it any lower than about one and one-half per cent. I think we specify two per cent for that material.

The volume of oil and water you have in the mixture shouldn't be great enough to quite fill the voids in the mixture after it is compacted, because as soon as that happens the mixture loses its stability. It seems there should be some account taken of the voids when you decide you can use so much or so little moisture in your mix.

Mr. Seifried:

I think that is important. We talk very loosely, of course, about a certain per cent of moisture, a certain per cent of this, that, or the other. From the remarks that have been made, we can see it isn't per cent of the volume of certain areas to be covered, because the surface area varies with the grading. I think we can leave it to the laboratories to tell us how much to use. When we get out in the field we can use it, and ''cuss'' them afterward if we use too much.

K. C. Vail:
Colorado

We inherited a specification. We had very little contract work on the oil, but this year we had a one per cent minimum. We fought very strenuously with the contractor, but never could quite make it. We got it down to about one and one-half. There were a few places in the road up around two and one-half. The road was put in with the paver, and we had a lot of hot, dry weather. It was at about 10,000 feet altitude above Echo Lake. In two or three weeks' time, where it was two and one-half, it came down to one and one-half after it was rolled, but not sealed.

Mr. Seifried:

The point is that we do lose that moisture if we don't seal it in and give it a little time to evaporate out. Mr. Groves, don't you have anything to offer?

H. G. Groves:
Montana

The Chairman put me on the spot on this fog coat proposition. That is probably a misnomer. After the prime coat has laid there, particularly if you are hauling gravel over your road, you deposit a certain amount of dust, and the idea was merely to create a tack or a bond between your mat and the base, by using a light application of road oil which amounts almost to a fog.

Mr. Seifried: In the past we have effected the same thing in a similar way.

Mr. Sedillo, have you anything further?

R. R. Sedillo:
New Mexico

Hearing about moisture, we have another problem over in New Mexico in not having any moisture at all. That hasn't been taken up. We have covered the cost of oiling a mile of road here and there, but there is another thing we have to consider, and that is the amount of material blown away from the road in a year.

In the last six months, where we finished a road and placed some crushed rock on it, inside of four months, due to a 50-mile per hour, or more, wind, there will be nothing left. As the traffic grinds the crushed stone the wind blows it away, so we have to oil them right after we build them.

In considering the cost of oiling, we figure it is more of a conservation cost than just the cost of oiling a mile of road, or whatever it is.

Mr. Seifried: Thank you, Mr. Sedillo. Gentlemen, we have done pretty well. I want to congratulate you, and thank you for your patience and attendance. You have certainly been fine and attentive.

I am sure the committee appreciates very much the excellent discussion we have had, and I feel we have accomplished to a large extent the purpose I had in mind when I started, for the field men to throw some of these problems back on the technicians, and I feel we have received considerable help. I wish to thank you very much.

The meeting now stands adjourned.

Papers Prepared

on

Design and Construction of Bituminous Surfaces

for

The Montana National Bituminous Conference

Seal Coats With and Without Covering

By

ART LOFORTH
Construction Engineer
Department of Highways, State of Nevada

The advent of light asphaltic road oil construction in Nevada dates back to 1927.

As climate governs the type of pavement suited to a locality it might be well to gage any statement to follow with the fact in mind that the average rainfall in Nevada is less than 10 inches. On a rainfall map of the United States it will be noted that the area shown in white indicating 10 inches or less covers practically the entire portion of Nevada, a part of southern California, a rather large portion of Arizona, and that part of Utah outlined as the Great Salt Lake Desert. This area then might be considered as a zone in which one set of standard specifications could apply. But, one other factor, the excessive range in temperature must also be considered. In northern Nevada, in Elko County, the thermometer has hovered near 60 below for a two week period in more than one winter. In southern Nevada the range is from about the freezing point to well over 100°. The area in white can be possibly extended to include some of the shaded portions of the map of the inter-mountain region denoting 15 inches rainfall. However, it is a certainty that those sections shown in black for a 70 inch rainfall such as the western Carolinas or the northwest coast region are definitely out.

The first roadmix section in Nevada was a five mile project constructed in 1927 by State forces, using an SC-2 road oil, then commonly called "60-70." The only information available when starting was received from the oil company representative successful in making the sale of the road oil. A specification written from his instructions would have read about as follows: (1) Use ½ gallon per square yard for each inch of thickness, which for a mat of three inches makes 1½ gallons, (2) Mix with blade, disc and harrow until all free balls of oil are positively eliminated, (3) Spread with blades and open to traffic. This was supposed to be all of the manipulation required as tire traffic was to compact the mat and provide a seal. It was presumed that the pumping action of rubber-tired vehicles would flush a small amount of oil to the surface providing an effective seal.

It was not until 1932 or five years later that any additional treatment was required on this section, nor any maintenance except for a rather excessive amount of patching along the edges of the comparatively narrow 18-foot surface. Probably ‘‘beginner’s luck’’ played an important part in getting such satisfactory results. In 1930, on a section constructed in 1928, the first seal coat was applied. Raveling and sloughing of the surface on this section gave indication that some treatment was necessary immediately to prevent complete disintegration. The remedy suggested was an application of ⅛ gallon of SC-2 road oil to the square yard. Due to inaccurate setting of controls on the distributor the application turned out to be 1/12 gallon to the square yard which proved fairly satisfactory, as this relatively thin film completely covered the surface giving sufficient seal to eliminate the raveling and sloughing.

Due to the apparent success of the first sealing specifications were written into all subsequent contracts for low type asphaltic surface construction providing an SC-2 seal. No definite time limit after completion of the surface for application of the seal was prescribed, this being left to the judgment of the Engineer. In some cases the application was made immediately after the final lay-down, in other sections the seal was applied after the mat had obtained full compaction. Experience indicated that the newly laid surface should be given to traffic for at least 10 days prior to the application of the seal.

The results obtained during the ensuing season were fairly satisfactory. However the tendency to slow drying and somewhat greasy and slippery conditions on some projects created a traffic hazard. This was particularly true on projects in the northern part of the State where it was necessary to apply the seal during periods of low temperature. The only remedy was to immediately sand the surface. Although the cost of sanding was not excessive it was believed that by using a faster drying asphaltic material this cost could be eliminated.

To meet such a requirement a medium curing product with a kerosene solvent was recommended and the following season, 1933, the first MC-2 seal was applied.

The slight increased cost over the SC-2 was negligible compared with the decided improvement which was immediately apparent. A desired amount of penetration was obtained; drying occurred within a comparatively short time, sanding was practically nil. This was considered the ideal material and for the five years to follow no other type was used.

During this period frequent suggestions and recommendations were made for the use of asphaltic material which would dry and lose its tackiness quicker than the medium curing asphalt. The requirements for such a material would necessitate the use of a rapid curing asphaltic type. The only deterrent was the highly inflammable naphtha solvent. Stories were told of such a seal becoming ignited and causing sheets of flame for the length of a full city block. A neighboring state using this type on an experimental job had flagmen assisted by police officers to stop all traffic with strict orders that no lighted smokes or burning matches be dropped on the sealed surface. The exact length of time that this precaution was to be followed is not recalled, but it might have been for a period of two hours or thereabouts.

Oil company representatives were hesitant to suggest a given temperature to which the RC material could be safely re-heated. These bugbears were finally dissipated and during the past two years RC-2 has almost entirely replaced the MC-2 for sealing. However, it is well to remember that considerable more precaution is required in re-heating material with a naphtha solvent, and while a retort with ordinary care is safe for an MC-2 material, steam units are required for the RC materials.

In the majority of jobs an application of ⅛ gallon of seal has been found to be a desirable amount and under normal weather conditions no covering of any sort is required except in places where travel might be inconvenienced. In such cases a blotter sand is applied in sufficient amount only to prevent picking up by tires. Ordinarily, or say on nine out of ten jobs, an application of ⅛ gallon will penetrate or dry sufficiently in two to six hours to be free of picking up, and during this drying period it is usually more satisfactory and economical to pilot travel through a dry lane than to sand. However, on a few of the more heavily traveled roads sanding is the rule.

Experience has been that with or without sanding and using an application not exceeding ⅛ gallon that a non-skid surface can be had.

Permeability tests on roadmix surfaces, or even on asphaltic concrete pavement, which will not show leaks does not necessarily indicate that a seal coat is not required. An application on those dense mixes will still assist to bond the top portion of the mat to prevent, or at least to lessen, raveling.

A bleeding seal cannot always be attributed to an excess amount of oil. On at least three projects constructed within the past few years bleeding developed on certain sections with the first warm day of spring. All three projects were road-mixed and sealed during the cooler weather in the fall, and heavy rains fell during mixing operations on these particular sections which bled. The deduction then would be that moisture caused the bleeding. Before final laying of the windrow laboratory tests were made for moisture and the results showed only between 1% and 1½% compared with the standard specification allowance of 2%. Past experience indicated that 2% is a reasonable limit. In addition to the windrow moistened with rain another possible contributing factor might have been a wet subgrade. Because of apparent dryness of the subgrade no tests were made for moisture and the possibility exists that some might have worked to the surface to affect the seal. Suspicion has been aroused on all of these three projects because adjoining sections laid down prior to the rain, and having the same quality of gravel, the same quantities of oil and seal, were not likewise affected.

Another source of bleeding results in plant-mix, and to a lesser extent on roadmix jobs, where the asphaltic material has been "cut back" and where in the agitating process there is improper distribution of the solvents. This has been definitely determined on several jobs, and especially with the medium cured types, where the solvent had not been sufficiently eliminated before laying. While such asphaltic materials can be a source of trouble in the mixes it would not ordinarily cause difficulty in the thin film of a seal coat where the solvent quickly evaporates.

Since 1927 Nevada has constructed 2,547 miles of asphaltic roadmix surfaces. Of this total 141 miles were reconstructed with a plantmix. Of the 141 miles of plantmix 97 miles were blade-laid and 44 miles machine laid. Of the 141 miles of plantmix 69 miles were sealed in the same manner as the roadmixes and without any covering, while the remainder of 72 was sealed with emulsified asphalt or MC-4 and covered with screenings.

SEALS WITH COVER

Application of a seal to be covered with screenings for the various asphaltic materials is specified to vary from 1/6 to ¼ gallon. Standard specifications for screenings measured on square openings are: 100% passing ½″, 95-100%, ⅜″, and 0-5%, No. 10. These screenings are applied at the rate of 12 to 15 pounds to the square yard and rolled into the asphaltic material before it cools or hardens.

The 72 miles were divided among seven contracts on which the appearance of the seal and screenings on three jobs can be rated as excellent, and on the remaining four from fair to poor. This rating is based mostly on the appearance of the screenings following a period of a month to two years.

The first four were sealed with penetration type emulsified asphalt, during September and October, 1937.

Two of these in the same district were sealed in cool weather close to a minimum specified temperature of 50°. These two are listed as poor due to "mangy dog" appearance caused by large blotches from which screenings kicked off. It is evident that with air temperatures at 50° the asphalt hardened too rapidly to allow the screenings to stick.

The third seal and screenings was manipulated in the same manner but at a temperature ranging from 60° to 70°. This surface is rated as good with very few bare spots and the warmer temperature possibly bears out the above statement that 50° is too low.

The fourth job was handled likewise with temperatures even slightly higher than 60° or 70°. Although the surface can only be rated as fair the unsatisfactory appearance is partly due to missing sprays, and partly to bare spots caused by the picking up under fast traffic, although the screenings were allowed to set three or four hours after rolling before opening to travel which ordinarily would seem long enough.

The outcome of the unsatisfactory appearance of the first two mentioned seals was to write an additional clause in the special provisions which provided that spreading of screenings shall follow immediately the application of the bituminous material and further provided that in no case should the bituminous material be applied more than two minutes in advance of spreading the cover.

The first project commenced this year specified an MC-4 applica-ion. The seal was placed with temperatures upwards of 60 degrees. The result is a rating of excellent.

A few short spreads on this project soon proved the impracticability of applying the seal for a distance limited to two minutes in advance of spreading the screenings. A length of 2,200 feet covering the half width of surface of 11 feet was adopted as a reasonable length and although portions were not covered in some cases up to 30 minutes there was no indication of screenings failing to stick.

The next two and last of the contracts to be completed this season were both plant mixes, the first MC-3 and 4, blade laid, the other SC-6 and machine laid, both sealed with emulsified asphalt and covered with screenings. Incidentally both were constructed by the same contractor. And further the same distributor and an excellent type of custom built chip spreader was used; and it might be added that the same inspectors covered both jobs. The first job is excellent having a stone covering with hardly a blemish while the last named can be rated no more than fair because of spots and streaks. From the foregoing information it is evident that the distinct variation of the two jobs cannot be attributed to equipment or the ''personal equation.''

Two theories advanced to account for this wide variation are: (1) the first job was covered during a moderate temperature ranging 60° to 75°; the latter, 85° to 100°; and as a result of this increased temperature the asphaltic seal remained soft too long to hold the screenings under fast traffic; (2) the surface of the first job being an MC-3 and 4 mix would stay softer longer than the one with SC-6 with the result that the screenings would receive a better bond by becoming slightly embedded under the roller.

CONCLUSIONS

As previously stated SC, MC and RC grade 2 are the only types which have been used for sealing asphaltic road surfaces without cover, with RC-2 most in favor at this time.

It may have been noted that the only types used thus far for sealing the plantmixes on which screenings were used were MC-4 and emulsified asphalt (90-95).

It should not be inferred that the above mentioned types are the only kinds recommended for sealing in Nevada. When this year's program was being prepared at least two other types of asphaltic material for seal with screenings were contemplated, SC-6 and RC-4 or 5. However, at this writing no projects have proceeded to the point where they are ready for these seals.

A proper seal depends upon the uniformity of the spread of materials. This uniformity cannot be obtained unless the right type of equipment is used.

Of all the kinds of equipment in the field of highway construction possibly no two pieces are prone to as much ''home made'' assembly as an oil distributor and stone spreader. And on all equipment used for constructing roads that should be required to give precision and positiveness these two are near the top of the list. Too often the operators are criticized for the poor application when the fault lies entirely in the equipment.

It is most disconcerting to ride along a highway and see long narrow streaks of surface raveling due to no other cause than a miss of a spray for lengths of a quarter of a mile, or even longer. And even a spray ejecting a deficient amount of oil which is not readily discernible at the time will sometimes develop into a raveling streak after a year or more.

It is believed that if a ''closed season'' were declared on most home-made distributors and stone spreaders, and deliveries were required from any one of at least a dozen established manufacturers some of whom for upwards of a quarter of a century have specialized in seal coat machinery—that the results would be gratifying and would be a display of efficiency. It would be a display of economy when the fact is realized that $1,000 is the penalty for reconditioning one mile of roadmix.

Design and Construction of Dense Graded Bituminous Mat Street Surfacing in Colorado Springs, 1932-1939

By

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Colorado Springs, Colorado

GENERAL

The construction of processed in place bituminous mat surfacing was initiated in September, 1932, when 13 blocks of streets, totalling 19,361 square yards, were constructed experimentally. As the cost per square yard was low and the results very satisfactory, the program has been expanded and continued from approximately May 15th to October 15th each year, until at the present time (August 15th, 1939) the total bituminous mat or oiled gravel surfacing laid on City streets and alleys amounts to 709 blocks of streets and seven blocks of alleys and has a total length of 61.74 miles and a total area of 1,435,355 square yards. In addition to the above, 49 blocks of streets (28,045 square yards) in Evergreen Cemetery, many private drives and parking areas most of which were for hospitals and other institutions (27,000 square yards), three airport runways each 4,600 feet by 150 feet and one taxiway 1,100 feet by 150 feet. (234,834 square yards), and 12.79 miles of roadway (146,562 square yards) in Palmer Park, and Garden of the Gods and North Cheyenne and Bear Creek Canyons, were surfaced in place with this processed in place bituminous mat. The grand total of all work done including the Cemetery, Airport, and Mountain Park roads amounts to 1,871,796 square yards or the equivalent surface area of almost 160 miles of roadway 20 feet in width.

With the exception of the approaches to new bridges, constructed since 1935 where the traffic is heavy and concentrated, all streets and alleys within the City and in Evergreen Cemetery were laid to a compacted thickness of one inch. The Airport runways and taxiway and all Mountain Park roads were laid to a compacted thickness of one and one-half inches. On both the one inch and one and one-half inch thickness work, the edges were constructed to a thickness of from two and one-half to three inches.

All of the above work was done by City Street Department employes and equipment with the exception of the Airport work and the Paseo Road to Palmer Park where WPA laborers and truck drivers were used, and on a portion of the Garden of the Gods work where the material haul was done by CCC laborers, truck drivers and trucks. The surfacing of streets and alleys in the City is petitioned for by the abutting property and when a sufficient number of these petitions have been filed, an Improvement District is formed and upon completion of the work the benefitted property is assessed at the rate of $0.15 per square yard. The City pays the cost of intersections. No bonds have ever been issued, the City paying its share and financing the property owners' portion until assessments are paid, out of its working capital.

Our reason for using the dense graded type of mat is that our local materials are only suitable for this type of work and our reason for processing in place, instead of plant mix, are that we do not own a hot mix plant and equipment for processing in place requires a considerably less investment than for the plant mix type.

FOUNDATION OR BASE

Foundation or base soil over about 50% of the area of the City is ideal for this type of surfacing, being a natural gravel or sand, but over the remaining about 50% of the area of the City the natural soil is an adobe clay which in most cases remains dry and stable. In the adobe territory all streets and alleys had in past years been surfaced with gravel to a thickness of from one inch to six inches. During 1933, 1934 and 1935 the demand for the bituminous mat surfacing was so great and our program so large that some streets were surfaced which did not have more than one inch of gravel covering the adobe clay. Some of these streets during the rainy summer or spring weather, when the ground water level is raised considerably, show base failures. This has been mostly caused by water coming to the surface through capillary attraction. These areas of base failure are small in comparison to the total yardage which has been laid, and the base stabilization work necessary to repair them is being done under maintenance. During the past four years the practice, when unsatisfactory base soil is encountered, has been to construct a gravel base at least four inches in thickness several months in advance of laying the bituminous mat. Most of our City work for the past four years has been confined to streets for which the property owners petitioned for the construction of concrete curb and gutter and oiled gravel surfacing. The concrete curb and gutter has been built by contract the same summer the Improvement District was formed and during the fall and early winter months the street is graded to conform to the new curb and gutter construction and if the base soil is not suitable the excavations are carried four inches (six inches along the gutter edge) below grade and during the winter or early spring months a gravel base is constructed. This gravel base is rolled and traffic compacted and the bituminous mat processed and laid on this base during the following summer. The curb and gutter is assessed to the abutting property but no charge is made for the grading and graveling which is done by the Street Department out of its appropriation for street grading and street graveling.

AGGREGATE

Specifications of the Colorado State Highway Department for Mineral Aggregate for Road Mix Oil Processed Surfacing call for the following grading:

Passing a sieve with three-fourths (¾) inch square openings	100%
Passing a sieve with three eighths (⅜) inch square openings, not more than	80%
Passing a No. 10 sieve	40% to 60%
Passing a No. 200 sieve (by washing)	7% to 15%

The City Street Department owns several gravel pits within or adjacent to the City and while the grading of this gravel does not comply with the State Highway Department specifications as to amount of material retained on the three-eighths (⅜) inch sieve, and is also deficient in the amount passing the No. 200 sieve, very excellent results have been obtained in constructing a dense graded mat by the addition of gold reduction mill slimes from the dumps of an old mill which is on property owned by the City Water Department. A sieve analysis of our average pit run gravel after the oversize is crushed is as follows:

Passing three-fourths (¾) inch sieve	100%
Passing three-eighths (⅜) inch sieve	82% to 90%
Passing a No. 10 sieve	38% to 54%
Passing a No. 200 sieve (by washing)	5% to 7%

An average sieve analysis of the mill slimes or dust which is hauled-in in sufficient quantity to make the required 200 mesh content of the mixed aggregate is as follows:

Passing three-fourths (¾) inch sieve	100%
Passing three-eighths (⅜) inch sieve	100%
Passing No. 10 sieve	100%
Passing No. 48 sieve	98%
Passing No. 100 sieve	93%
Passing No. 200 sieve (by washing)	85%

When the pit run gravel is mixed with the windrow of gravel salvaged from the original gravel street surfacing and sufficient mill slimes have been added to bring the 200 mesh content of the total aggregate to 13%, an average sieve analysis of our mineral aggregate which is processed with the asphaltic oil is as follows:

Passing a three-fourths (¾) inch sieve	100%
Passing a three-fourths (¾) inch retained on three-eighths (⅜) inch sieve	13%
Passing a three-eighths (⅜) inch retained on a No. 10 sieve	40%
Passing a No. 10 retained on a No. 48 sieve	25%
Passing a No. 48 retained on a No. 100 sieve	5%
Passing a No. 100 retained on a No. 200 sieve	4%
Passing a No. 200 (by washing)	13%

The State Highway Department specifications call for at least 20% of the entire aggregate to be retained on a three-eighths (⅜) inch sieve, but this is intended for mat construction having a thickness of from two inches to three inches. On our one inch thickness mat this amount of material retained on a three-eighths (⅜) inch sieve would make a very coarse graded rough surfaced mat.

Until the beginning of the 1938 season all gravel was hand loaded, but during the winter of 1938 a gravel loading, screening and crushing plant was assembled from used parts and built in the Street Department shops, and the operation of this plant during the 1938 season has cut the cost of gravel delivered on the streets with an average haul of about two and one-half miles, from 60 cents per cubic yard to 35 cents per cubic yard. The capacity of this plant is about 400 cubic yards in eight hours. When gravel was hand loaded it was also necessary to rake the oversize material out of the street.

ASPHALTIC OIL

Practically all asphaltic oil used on this work has been (Asphalt Institute Designation) SC-3 or slow curing oil having an asphalt content of not less than 65% nor more than 75%. Cut back asphalt designation MC-3 has been used on several streets but results obtained have not been noticeably different from those obtained from the SC-3 and we have not felt justified in going to this additional expense. Cut back asphalt RC-2 and RC-3 has been used on all seal coat work, but this seal coating is not done under construction but under maintenance usually from three to five years after the original mat was laid.

Specifications, together with an estimate of the amount required for the year, are prepared and sent to the various oil companies manufacturing this material in March or April of each year and contract awarded to the low bidder. So far our supply has been a negative spot material from the black oil fields of Wyoming. Our specifications are the same as those of the Colorado Highway Department and the United States Bureau of Public Roads.

CONSTRUCTION METHODS

Assuming that the street is to the proper crown and grade and that the base conditions are satisfactory, the first step in construction is the cutting out with a motor grader along the edge of the concrete gutter, or if the street or road is not curb and guttered and a 20, 30 or 40 foot strip is to be constructed, the cutting out of the staked edge to the edge thickness of from two and one-half to three inches. The motor grader also blades the entire width cutting out any minor irregularities and corrugations and with the material from the edges, the cut material and all loose unpacked gravel on the surface makes a windrow in the center of the street. This windrow is measured every 100 feet and the volume per lineal foot computed. The total aggregate required to make the compacted thickness of the mat is computed to which is added the extra amount necessary to make the thickened edges, a sufficient quantity to be left after laying for a maintenance ridge, (about .25 cubic foot per lineal foot for average widths) and also the amount required to lay alley and street intersections. If the compacted thickness is to be one inch the amount of gravel required is taken as the number of square feet of surface area multipled by .10 foot plus the volume required to make the thickened edges and the volume for the maintenance ridge less the center windrow of gravel salvaged from the old gravel surfacing. If the compacted thickness is to be one and one-half inches the area in square feet is multiplied by .15 foot. One and one-half ton dump trucks loaded to 65 cubic feet each are used in the gravel haul and from computations as outlined above, are dumped on this wondrow of salvaged gravel so as to leave the necessary amount of gravel per lineal foot. Stakes are set in the windrow of salvaged gravel defining the length on which the 65 cubic feet truck load of pit gravel is to be dumped. Samples are taken of both the gravel salvaged from the street, and from the pit gravel trucked in, and from a sieve analysis of these combined in their proper proportions, the deficiency of fines or material passing the 200 mesh sieve is determined. The windrow of gravel is then staked for the proper dumping of the fines. This windrow consisting of gravel salvaged from the street, pit gravel trucked in, and mill slimes, is now dry mixed by the motor graders until the moisture content is not to exceed one per cent by weight. The next step is the brooming of the base by a power broom to remove all dust ready for the hot (175° F. to 200°F.) SC-3 base prime coat applied at a rate of .15 gallon per square yard by the distributor. The entire base is covered by this prime or tack coat, the windrow of aggregate being moved by the motor graders on to the prime coated portion to allow the remainder of the base to be broomed and prime coated. There is no doubt that the SC-1 or 2 oil or MC-1 cutback would be more suitable for the tack or prime coat but limited storage facilities prevent us from using them. From a sieve analysis of a composite sample of the aggregate the amount of the SC-3 oil required is computed. There are several formulas for this, the principal ones being those of the New Mexico and Wyoming Highway Departments which are as follows:

New Mexico

$P = 0.02a + 0.07b + 0.15c + .2d.$
P = % of oil by weight to dry weight of aggregate.
a = % of aggregate retained on No. 48 sieve.
b = % of aggregate passing a No. 48 and retained on No. 100.
c = % of aggregate passing a No. 100 and retained on No. 200.
d = % of aggregate passing a No. 200 sieve.

Wyoming

$P = (0.015a - 0.03b - 0.17c)\ 1.4.$
P = % of oil by weight to dry weight of aggregate.
a = % of aggregate retained on No. 10 sieve.
b = %of aggregate passing No. 10 retained on No. 200 sieve.
c = % of aggregate passing No. 200 sieve.

From our average composite sieve analysis which is given in detail under aggregate, the oil content from New Mexico formula figures 5.11% and from the Wyoming 5.63%. We run our aggregate sieve analysis through both formulas and use 92% of the mean which in this case would be 4.94%. The reason for this is that when these formulae were developed the amount of 200 mesh material was obtained by dry sieving. As our composite samples are more or less variable, our oil content runs from a minimum of 4.8% to a maximum of about 5.2%. We also vary the oil content slightly according to traffic conditions, using a leaner mix on heavy traffic and a richer mix on heavily shaded and light traffic streets.

After the computation of the amount of oil is made, it is reduced to the number of spreads over the length of the street with 10 foot spray bars and at a rate of 0.2 gallons per square yard. A final spread with a five foot bar at a 0.2 gallon rate or with a 10 foot bar at a 0.1 gallon rate, is sometimes given to make the exact gallonage. The distributor operator is given a memorandum of the total gallonage, corrected to 60° F. temperature, and the number of spreads and rate per square yard.

The dry aggregate is spread over from one-fourth to one-half of the width of the street and the application of the hot SC-3 oil (150° F.-175° F.) is started. One motor grader follows immediately behind the distributor turning dry aggregate over the hot oil. This is continued until the entire amount of oil is applied when mixing or processing with two or more motor graders is commenced. This mixing is continued until the entire mass is of uniform color and texture and contains no balls of oil and fines. If rainfall occurs during the processing, the partially mixed oil and aggregate is windrowed as quickly as possible and kept in a windrow until the rain is over and the base thoroughly dried. Processing is then resumed and continued until all moisture is worked out and the mass uniformily mixed. Upon the completion of mixing, one motor grader is used laying, this being done in light lifts from the windrow which has been moved to one side of the street, until there only remains enough material in the windrow for the maintenance ridge and that required to build the intersections. The material for the intersections is hand loaded from the ridge, trucked to place and spread with a motor grader.

Immediately following the laying of the mat, rolling with a nine-wheeled pneumatic tired roller pulled by a one and one-half ton dump truck is started at one gutter or edge and continued to the other gutter or edge, lapping about two-thirds (⅔) of the roller width each trip. After rolling, a light center control motor grader picks up the remaining maintenance ridge and with a light blade pressure only sufficient to cut off high places,carries the ridge across the street and back again. This maintenance is necessary daily until a large percentage of the volatile oils are evaporated and a set has taken place. This usually takes place in two or three days depending on weather and traffic conditions. Traffic is not interrupted at any time during the work, although pedestal SLOW warning signs are used until the work is completed, and, of course, the windrow or ridge is protected by red lanterns or flares during the night.

TRAFFIC COMPACTION

If the finished street is wider than 30 feet, barricades 16 feet in width are placed forcing the traffic to the edges for a sufficient length of time to thoroughly compact the edges, and if the street is 50 feet feet or more in width the barricades are moved after the edges are compacted, and placed so that the center of the street only is open to traffic. When the barricades are placed so as to force all traffic to the edges, temporary NO PARKING signs are erected back of the curbing. This barricading is only done during daylight hours.

DESIGN AND CONSTRUCTION IMPROVEMENTS

During the past year we have improved our aggregate grading by increasing the amount retained on a 3/8 inch sieve from 5% to 10% to 10% to 15% and decreased the amount passing the number 10 sieve from 50% to 60% to 40% to 50%. This makes a mat which should have considerably more stability and gives a very good non-skid surface texture.

Construction improvements have been a closer control of our aggregate tonnage by weighing the truck loads at least once daily and the purchase and use of a spring steel wire brush which is attached to the moldboard of the finishing motor grader and used on the final maintenance of the street. With the coarser aggregate used this fills the voids in any surface coarse spots with fines.

EQUIPMENT

The equipment used by the City Street Department on the construction of this work and which has a good weather production capacity of about 125,000 square yards of mat one inch in compacted thickness per month, is as follows:

1—Model 200 Municipal Supply Co. Bituminous Distributor mounted on a Model T51 GMC Truck.

1—1,200 gallon capacity insulated tank oil trailer (used only on Airport and Mountain Park work).

8—1½ ton dual rear wheel trucks with hyraulic dump bodies. (These trucks also do aggregate haul).

1—Model 65 Caterpillar Tractor with Model 64 Killefer Rotary Fresno Scraper (Used at Gravel Plant during 1938 and 1939).

1—Gravel screening and crushing plant with 15 cubic yard bin. (Used only during 1938 and 1939).

2—Model 77 Austin Tandem Drive Motor Graders with 10 foot oil mix moldboard.

1—Model 151 Adams Tandem Drive Motor Grader with 12 foot oil mix moldboard.

1—Model No. 11 Caterpillar Diesel Auto Patrol with 12 foot oil mix moldboard.

1—Wehr Fordson center control motor grader with 8 foot moldboard.

1—9 wheel pneumatic tired roller.

1—9 foot width Buckeye Sand and Chip Spreader. (Used only on maintenance seal coat work).

CONCLUSION

The Bituminous Mat surfacing which has been laid on our streets, Airport and Mountain Park Roads during the past six years has been entirely satisfactory both from the City's and the taxpayer's standpoint. Its low cost, freedom from dust, smooth riding surface under all weather conditions, and its ability to carry large storm water flows without scouring the gutters, has given almost 100% property owner and taxpayer approval. From the City's standpoint, the maintenance to date has been much lower than the continual gravelling, sprinkling and smoothing of the gravel surfaced streets.

Design and Construction of Bituminous Surfaces In Cities

By

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In the past, practically all of the bituminous design and construction described has covered highways. It is my thought that the present trend in cities, particularly those having a small population, is toward a cheaper type of pavement and they would be interested in design and construction of bituminous surfaces for their streets. I intend to cover two types of bituminous surfaces used in two cities in North Dakota.

The City of Bismarck, North Dakota, recently completed sixteen thousand square yards of two and one-half inch oil mix surfaces using a heavy Slow Curing road oil in a stationary plant, this work being done under P.W.A. regulations. The contract was let in the fall of 1938 and the work completed in the spring of 1939. The following are the unit prices and quantities of the low and successful bidder:

Item	Description	Quantity	Unit Price
No. 1	Earth Excavation	3,100 cu. yards	$ .70
No. 2	Preparation of subgrade	16,000 sq. yards	0.08
No. 3	Adjusting Manholes	20	10.00
No. 4	Adjusting Catchbasins	10	10.00
No. 5	Straight Curb and Gutter	4,071 Lin. Ft.	0.98
No. 6	Circular Curb and Gutter	510 Lin. Ft.	1.08
No. 7	Concrete Driveway	530 sq. yards	2.25
No. 8	2½″ Oil Mix in Place	16,000 sq. yards	0.50
No. 9	4½″ Compacted Base in Place	16,000 sq. yards	0.31

I will endeavor to cover the construction methods on each of the above items in chronological order.

EARTH EXCAVATION

The Contractor used a motor patrol with a scarifier attachment and scarified to a depth of approximately two inches, which brought the earth to approximately the desired grade. He then moved the material into windrows, filled any low places, and picked up the excess earth with a Barber Greene loader, hauled it to the destinations as specified by the Engineer.

PREPARATION OF SUBGRADE

The Contractor used a motor patrol, graded and rolled the subgrade to the elevations and sections as designated by the Engineer.

CURB, GUTTER AND DRIVEWAYS

In all concrete items the materials used met standard specifications for sand, gravel and cement.

Washed sand and gravel was shipped from a plant at Schmidt, North Dakota. Universal Atlas cement was used. The concrete was mixed in the following proportions: One part of cement, two and one-half parts of sand and three and one-half parts of gravel. A standard two-sack mixer was used with a spout that carried the concrete directly to the forms. The following are some typical results on seven-day compression tests made on six by twelve concrete cylinders at the age of seven days: 3,700 and 3,180 pounds per square inch.

ADJUSTING MANHOLES AND CATCHBASINS

All manholes and catchbasins were adjusted to grades as established by the Engineer under the usual construction methods.

CONSTRUCTION OF BASE COURSE

The Contractor obtained his surfacing gravel material from a pit located a few miles South of Bismarck. The following is a typical graduation of the base course gravel without binder added:

Sieve	% Passing
1½″	100
¾″	93.4
No. 4	69.0
No. 10	53.1
No. 40	19.6
No. 200	1.4
Weight per cu. ft.	117 lbs.
Physical Character	Good
Shale	None
Plasticity Index	None

This material was hauled to a stationary plant having a twin pug mill type of mixer and stockpiled. The soil binder used was obtained from a pit located on Ninth Street between Avenues C and D and had a typical grading as follows:

Sieve	% Passing
No. 10	100
Weight per cu. ft.	76 lbs.
Liquid Limit	59.3
Plastic Limit	22.0
Plasticity Index	37.3

This soil binder was also hauled to the stationary plant and stockpiled. These two materials were then mixed in the following proportions: Six pounds of clay to one hundred pounds of gravel with sufficient water to facilitate laying operations. A typical grading of the combined materials is as follows:

Sieve	% Passing
1½″	100
¾″	95
No. 4	72.7
No. 10	54.9
No. 40	31.4
No. 200	8.3
Plasticity Index of Fraction Passing a No. 40	5.0
Liquid Limit	21.8

After the gravel and soil binder were mixed in the above proportions with sufficient water they were hauled to the subgrade which had been previously prepared and dumped in windrows. Motor patrols were used in bringing the windrow of gravel and soil binder mixture to a uniform section, after which the subgrade was given an application of water in just sufficient quantity to thoroughly moisten the surface. The windrow of gravel and soil binder mixture was then laid to the section as specified on the plans by a motor patrol, after which it was rolled with a pneumatic tired roller until thorough compaction was attained. Then a smooth steel surface roller was used to obtain a smooth surface on the street section. This compaction was continued until a density of one hundred thirty-five pounds was obtained. The street was then barricaded and ready for the bituminous surface.

PRIME COAT

A prime coat of MC-1 at the rate of one-fourth gallon per square yard was applied with a distributor and allowed to thoroughly penetrate before any mixed material was hauled onto the street.

A typical analysis of the MC-1 used is as follows:

Viscosity, Saybolt Furol, at 77° F.	96
Flash Point Tag Open Cup	135° F.
Distillation, % by volume:	
Temperature	Per Cent off
437° F.	3.75
500° F.	29.5
600° F.	44.5
680° F.	49.0

Tests on Residue

Penetration at 77° F.	201
Ductility at 77° F.	100
Soluble in CCl_4	99.8

Gravel for the oil mix surface was hauled from a pit located near Bismarck and stockpiled at the stationary plant. A fine sand was also hauled from a pit near Bismarck and stockpiled at the plant. This material was to be used to increase the percentage of two-hundred mesh material in the gravel, which was deficient in that particular sized material. Consequently, a very small amount of this last material was added to the gravel before it entered the rotary drum drier.

These two materials were combined in the proper proportions and taken by a bucket conveyor up into the drier. After passing through the drier, they were elevated to a screen and a separation made of the material. That fraction passing a three-fourths inch sieve was placed in one bin, and the fraction passing the one-fourth inch sieve was placed into another bin.

The aggregate was given sufficient heat when passing through the drier to always keep the surface moisture below one per cent, as required in the specifications. It was necessary to hold the temperature of the aggregate at about 220° F. in order to reduce the moisture content below one per cent.

The material from the two bins was then recombined by weight in such proportions as to give the following typical gradation:

Sieve	% Passing
¾″	100
⅜″	79.4
¼″	69.1
No. 8	49.9
No. 10	44.9
No. 20	34.4
No. 28	28.0
No. 48	18.3
No. 80	10.6
No. 100	9.7
No. 200	5.0

This material was weighed and dropped into a twin pug mill type of mixer to which was added four per cent by weight of SC-6 road oil. The capacity of the mixer used was two thousand pounds. The gravel and oil was mixed at least thirty seconds, or until thoroughly mixed and of a uniform color. From the mixer, it was dumped into trucks and hauled to the street on the previously premixed base.

The oil mix was dumped in windrows on the street, the windrows being brought to uniform section by a motor patrol before being laid to the section as required on the plans.

After the oil mix was laid to section, it was thoroughly rolled with a steel roller until compact. The street was then opened to traffic for several days before the seal coat was applied.

A typical analysis of the SC-6 road oil used is as follows:

Viscosity, Saybolt Furol, at 140° F.	479
Flash Point, Cleveland Open Cup	240
Loss, 50 gr. 5 hrs, at 325° F.	4.2
Per cent Residue 100 Penetration	80.3
Per cent Soluble in CCl_4	99.8
Ductility of Residue at 77° F.	100+

SEAL COAT

After the street had been open to traffic for two or three weeks, it was thoroughly broomed and all deleterious material removed. The street was then given an application of RC-2 at the rate of one-fourth gallon per square yard, immediately followed by an application of coarse sand at about twenty pounds per square yard. This coarse sand was applied by a mechanical spreader. The sand was then broomed to secure a uniform spread of sand over the entire surface and immediately rolled with a steel roller.

A typical analysis of the RC-2 used is as follows:

Viscosity, Saybolt Furol, at 122° F.	375
Flash Point, Open Tag	85° F.

Distillation:

Temperature	Per Cent off
374° F.	2.25
437° F.	12.75
500° F.	16.75
600° F.	21.50
680° F.	23.75

Tests on Residue

Penetration at 77° F.	96
Ductility at 77° F.	100+
Soluble in CCl_4	99.9

The streets resulting from the above construction appear to be very satisfactory and the residents are well pleased with the low cost surface. It is my opinion that this type of construction will prove adequate for almost any residential section having similar conditions.

ELLENDALE, NORTH DAKOTA

I will now give a brief outline of another type of bituminous construction used in Ellendale, North Dakota, using a heavy Medium Curing cutback in a traveling plant, the work also being done under P.W.A. regulations. This project consisted of 55,000 square yards of stabilized gravel base four inches thick and a bituminous mat three inches thick; also 25,276 square yards of stabilized shoulders three inches thick.

The existing streets were excavated to grade and the excess earth deposited as directed by the Engineer.

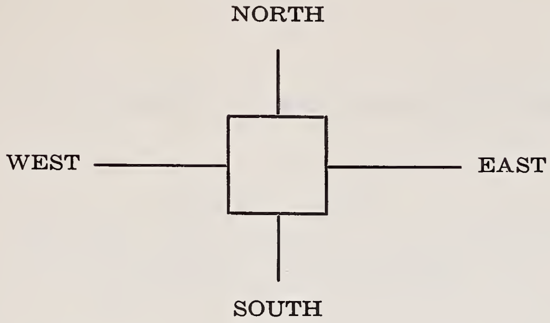
A stabilized gravel base four inches thick and twenty-four feet wide, which width did not cover the entire street section as, in a good many places, there were no existing curbs nor gutters. The gravel and soil binder were hauled separately in such quantities as to give the desired mixture meeting the same specifications as given above. These materials were then windrowed and a Barber Greene traveling plant was used to pick up and mix them thoroughly. During the mixing, sufficient water was added to permit the laying and compacting of the gravel and soil binder mixture. A pneumatic tired roller was used for the initial compaction, followed by a smooth steel roller to give the final compaction.

After the base had dried sufficiently, it was given a prime coat of MC-1 at the rate of one-fourth gallon per square yard. Immediately after the prime coat had been applied, the entire base was given an application of sand in just sufficient quantity to eliminate any tracking into business places and residences by pedestrians.

The material for the oil mix mat was then deposited on the street in two separate uniform windrows. The reason the gravel was placed in two windrows was that sufficient quantity was placed on the street to give a finished mat thickness of three inches, and by placing it in two windrows, it was a little easier to handle. An MC-4 cutback asphalt was the bituminous material used. The same Barber Greene traveling plant was used to mix this material as was used on the stabilized gravel base. It was found that the mixing was very difficult during the early hours in the morning or if the weather happened to be cool during the day. After the bituminous material and gravel were thoroughly mixed the two windrows were moved into one uniform windrow, the base swept and any excess sand that had been used to blot up the prime coat removed. The base was then given a fog coat of MC-1, possibly about one-twelfth gallon per square yard, after which the bituminous treated gravel was laid to the section called for on the plans, and rolled with a five-ton smooth steel roller.

Stabilized gravel shoulders three inches thick were placed on the street sections where no curb and gutter existed. These stabilized gravel shoulders were placed in much the same manner as the gravel base and given a prime coat of MC-1 at the rate of one-fourth gallon per square yard, immediately followed by an application of sand in sufficient quantity to blot up the excess oil and to prevent tracking into residences.

In laying the streets, an unusual practice was followed, which I believe is something that should be considered when two intersecting streets are being laid. When a street was laid East and West and the street North and South was to be laid, a "V" section was cut on the North and South side of the previously laid intersection extending from the corners of the street to the middle on each side. The material removed from this portion was then mixed with the material on the North and South street and the entire street and balance of the intersection was laid at one time. Just previous to laying, the edges of the "V" section were given a mop coat of MC-1. An illustration follows:



In this manner the Engineer was able to obtain an intersection that drained easier and get a section that did not have any feather edges when laying.

After the bituminous mix had been thoroughly rolled it was given a seal coat of RC-2 at the rate of one-fourth gallon per square yard, followed immediately by an application of coarse sand at the rate of approximately twenty pounds per square yard. This sand was broomed to uniformity and immediately rolled. After the sand had been in place three or four days, it was broomed again and all excess removed.

The costs of this project are as follows: $0.0853 per square yard, which includes a bituminous mat 3″ thick, a stabilized gravel base 4″ thick, stabilized gravel shoulders three inches thick and approximately three feet wide and necessary earth excavation.

It is my belief that a number of cities could take advantage of these types of surfacing and have adequate paved streets at a very low cost.

Design and Construction of Bituminous Surfaces

By

D. M. FAIRES
Maintenance and Construction Engineer
Idaho Bureau of Highways

The Design and Construction of Bituminous Surfaces is, of course, too broad a subject to be treated in detail in a paper of this kind. The most that can be done is to report the methods being used which give promise of being definite improvements over former methods and also to comment briefly on what seems to be the trend in designing and building bituminous surfaces. Strictly speaking, it is doubtful if there has been any marked improvement in this field for many years, but it is presumed the subject under discussion is confined to those surfaces in which the cementing medium consists of road oil or liquid asphalt.

Ten or twelve years ago, when Idaho first began using road oils, the purpose of their application was to reduce maintenance costs, prevent traffic erosion of the gravel surface and eliminate as much as possible the dust nuisance. With these objectives in view, the oils selected were those of low viscosity which would penetrate quickly. The cost of such surface treatment varied from five hundred to seven hundred dollars per mile and were often applied without any special preparation of the roadway. The short life of these surface treatments quickly led to the adoption of the road mixed oil mat, the aggregate usually being obtained by scarifying the existing surface. The development of oil mats has progressed steadily from this stage and, under our present standards of construction, a road mixed mat two and one-half inches thick and twenty-two feet wide costs about thirty-five hundred dollars per mile. A plant mixed mat of the same dimensions will cost five thousand dollars, or more, per mile. Thus, what began as a low cost, intermediate type of bituminous surface is rapidly being advanced to the class of a heavy duty paving. Except where traffic is extremely heavy, it seems doubtful if present construction standards should be further advanced until our primary and secondary systems are more nearly complete. We still have many hundreds of miles with surfaces much below the present standards.

A durable bituminous surface presupposes an unyielding foundation course of durable materials properly shaped and compacted, and this phase of construction will, no doubt, be competently discussed by other committees. Our reference to it is for the purpose of explaining that the practice in Idaho is to place both the subbase and the gravel or crushed rock surfacing the full width of the subgrade. The subgrade is dressed to a crowned section having transverse slopes of 1 in 48. This usually provides adequate run-off for surface water which finds its way to the subgrade, although there is a growing belief that a steeper slope would be an improvement. This practice of placing the subbase and surfacing the full width of the subgrade gives rise to the problem of how to treat the shoulders so they will carry normal traffic and still permit free drainage of water which may accumulate under the roadway. Our present practice is to roll the shoulders, usually with a loaded truck, but there is some danger of obtaining a degree of compaction that will seal off the drainage at the subgrade line. It is important that the surface of the shoulders be as nearly impervious as possible, to avoid saturation along the edge of the mat. Extending the mat to the full width of the roadway, or a penetration treatment of the shoulder with a suitable liquid asphalt, seem logical suggestions for correcting the present practice. This would eliminate what is now a very troublesome maintenance problem.

However, the adoption of a full-width bituminous surface would create a traffic hazard of major importance. The difference in color between the shoulder material and the oil mat is of great value in defining the travelled way, especially at night. No doubt it would be necessary to define the travelled way by the use of an outside traffic stripe or by spraying the outside edge of the shoulder with a light coat of white or yellow paint.

One of the minor changes in construction which has been adopted during the past year is the omission of the thickened edges on the mat. The material formerly used in the thickened edge is now spread uniformly, giving the mat an additional foot of width. The theory behind this change is that the vehicles will not travel so near the edge and ravelling will thus be greatly reduced. Observation indicates that many vehicles, especially the heavy freight trucks, habitually use the extreme outer edge of the mat and it is doubtful if the change represents any improvement over our former practice.

We find MC-3 or SC-3 suitable for most of our construction, but heavier grades are frequently used in plant mix jobs. Mixing plants of either the traveling type or the stationary pug mill type afford an opportunity for much more accurate control than can be attained on the road mixed job, but better riding qualities have been uniformly obtained by the road mix method. Under favorable weather conditions, plant mixes using SC-4 and an aggregate not too densely graded can be manipulated on the road and laid down in the same manner as a road mix.

A project is now under construction on which an SC-6 is being used, which, of course, will necessitate the use of a mechanical spreader to handle the mix immediately on delivery from the plant. This project will be watched with considerable interest as we have not heretofore used an asphalt of this grade. The project lies at an elevation of 6,200 feet and the extreme range in temperature is probably 160 degrees. It will be interesting to observe if the mat has sufficient elasticity to withstand these extremes of heat and cold.

We do not consider a bituminous mat complete until it is properly sealed. For seals, an RC-4 or heavier is usually used, and this year we have used a 151-200 penetration asphalt on some jobs. Unless the cost is excessive, a cover coat of material passing a 5/8″ screen and retained on a 1/4″ screen is to be preferred.

The trend in Idaho is definitely in the direction of a more rigid control of all processes and toward a more carefully graded aggregate. Heavier grades of liquid asphalt are being tried out in different parts of the State. In attempting to reach a decision on the most suitable grade of liquid asphalts the advantages of the lighter grades should not be overlooked. Mats in which the lighter grades are used are more easily repaired by means of power equipment and, until all subgrades and foundation courses are perfectly constructed and free from defects, this will continue to be an important consideration on maintenance. There is some doubt as to the wisdom of using the grades which exclude machine maintenance unless the traffic load makes their use imperative.

In looking back over the years, we find that we have obtained some excellent results and some very poor results under what seem to be almost identical conditions. Our laboratory technicians have done much to set up uniform methods of control and to remove the element of chance. Nevertheless, we still find wide variation in results obtained under the same specifications, working with materials having the same physical characteristics. It is quite possible that we have, to some extent, overlooked the human element. There is a skill, a facility, in the handling of bituminous mixes that can be acquired only by actual experience under a variety of conditions. To know just what to do and just when to do it is a kind of special knowledge that can be obtained only on the job.

Design and Construction of Bituminous Surfaces In Mississippi

By

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Research Engineer
Mississippi Highway Department

For several years the writer has observed the trend in bituminous construction becoming more highly technical each year, and has done a great deal of work along the line of proving that the ranges in all phases are so wide that there is no need for such technicalities.

After five years devoted largely to this activity, with the thought of keeping practically everything on a simple, economical basis, I am prepared now to express my conviction that only common-sense principles, with field control, are necessary to insure success, some of them, of course, being contrary to some of the "old line" principles that came down to us many years ago based on experience on street paving work.

With no thought of reflecting in any way on the valuable work that has and is being done along technical lines in the laboratories, and with no thought of dictating to others the methods they should use in their field, I desire to discuss briefly the conclusion that I have reached with reference to the many controversial phases of bituminous construction, as applied to materials, soils, climate, etc., found in Mississippi.

One of the first of these is that we do not need the high stabilities for which we strived a few years ago! The high-speed, low-pressure tires of today, in most cases, will permit low-stability surfaces on adequate bases to render very satisfactory service.

Again, I have observed engineers impose very strict requirements on the refineries as to the penetration on a particular project, even going so far as to break up the range by specifying AC—, but stay "on the high side," or "on the low side." Early in the summer of 1938 we started with a high-stability aggregate and an 85-100 penetration AC, in a hot-mix, and by successive steps, in tank-car lots, worked on up to 300 penetration! These steps were always as much as ten points, and as we got to the lighter grades, the steps were by 50 points! Even during the peak of the summer temperatures it was difficult to observe, under traffic, any difference between 10 or 15 points in the heavier grades, or 30 to 50 points in the softer grades, and the 300 penetration did not seem to be too soft! A similar condition seems probable with liquid asphalts, although we have not yet reached such high penetration ranges!

In a discussion at the Glacier Park meeting in 1936 I made the statement that my observation of Mississippi work showed that we were using too hard an asphalt in everything.

In a paper delivered in 1937, one of our leading engineers proposed a simple rule, to-wit: "Use as much asphalt as possible, and as soft as even 85-100!" I took exception to the first part of his statement at that time, on the grounds that I thought that high percentages of asphalt reduced the stability, which automatically caused many engineers to lower the penetration to offset the loss in stability!

My remark at Glacier Park seemed to cause quite a bit of discussion, but our work during three summers since then has proven that statement to be correct, and it now seems very clearly indicated that the necessary stability can be had with the proper shape of aggregates, with a common sense, rather than highly technical gradation, and almost any bitumen that will keep the water out, hold the aggregate particles down, and not separate them to such an extent that the mechanical stability is broken up, will be satisfactory.

The experiences of five years work here briefly show that we can build successfully using the same line of thought that we use in our concrete paving!

The discovery that asphalts do harden on exposure has further complicated the analysis of the problem, opening up a vast program of argument as to "thick films vs. thin films!"

During the summer of 1939 we have worked our way up on the percentages of bitumen in a given gradation in a hot-mix, and now, during our hottest weather of the year, we are using 7 gallons more per ton of mix than we thought sufficient at the beginning of the summer, yet we have not suffered a serious loss of stability, even with steel-tired traffic!

We have also done a lot of work relative to the percentage of fines! We have a number of equally satisfactory experimental projects of the 1935 and 1936 seasons, some that were high in fines, some with medium amounts, and some with none! Here, too, a close study of our results of five seasons seems to indicate a wide range between success and failure.

It is interesting to look back over the large number of actual experimental projects that have been in service for periods ranging from one to five years, over lengths ranging from one-half mile to five miles, on all kinds of soils and bases, (some with no base other than the soil in the road, with no stabilization or treatment of any kind)! Where native materials were used "as is," or where imported aggregates of various kinds were blended in every conceivable way with the native material; where the bitumen was varied from 100% liquid asphalt, in the large assortment of make-up in which this material is available, through the blends of part liquid asphalt and part asphalt cement, to the straight hot-asphalt and hot-mix methods; where Mexican and Venezuelan crudes were placed alongside the ''Domestics''; and where the question of so called ''stripping'' of the bitumen in connection with certain aggregates has been closely studied, and certain conclusions seem to be justified.

A large number of projects embraced varying thicknesses to thoroughly study the argument as to thin or thick mats! On one project in particular, mats were built as thick as 24″, on super-elevations and grade-raises! New mats, widening mats, re-surfacing old pavements with thin mats, and timber bridge floors have all been covered in numerous ways and combinations. As we near the end of 1939, or fifth season of such work in the State of Mississippi, I do not think that I will be out of order when I advance a few simple suggestions, based on these varied projects, that should enable any one with common ''horse sense'' and a reasonable knowledge of asphalt as a binding medium, to build satisfactory surfaces, to-wit:

Use that aggregate that has the highest inherent, mechanical stability available within a reasonable economy range!

Do not destroy this inherent stability by breaking off the corners with too much heavy rolling!

With a high inherent stability, the permissible gradation ranges are much wider than where the natural stability is low!

High stability in the aggregate is much more dependable and durable than is the stability given to the mat through the use of hard bitumen!

High stability in the aggregate permits the use of relatively high percentages of high-penetration asphalt, which should prove to be more durable than lowered percentages of hard asphalt!

High stability in the mortar can be nullified by using too much low-stability coarse aggregate!

Low stability in the mortar can be improved by using a reasonable percentage of high-stability coarse aggregate!

It is logical that it is more difficult to build a road with a bunch of marbles than with a bunch of angular, interlocking particles!

Where aggregates are suspected or known to be hydrophilic, either the RC-1 grades of liquid asphalt or hot-mix with straight AC's will prove to be more satisfactory than any of the medium or slow-curing liquid asphalts.

Reasonable densities are important, but there does not seem to be evidence to support the high densities that some seem to demand, so long as the surface is dense and water-tight. This is easily secured by the use of proper sealing methods.

The gradations can be determined by economy and availability in the field! It is a natural consequence of all our experience in everything that honey-combed material is not what we should have, but there are many gradations using the same basic material that will give satisfactory service, and by trial we may find the gradation that will permit the smallest amount of bitumen to do the work we want it to do at the least total cost!

Thick mats have their place, as well as thin ones, but each must be designed to fit! The intermediate thicknesses between 1″ and 4″ do not seem to have as wide a range of usefulness as some think!

Thick mats (4″ and up) do assist the base!

The penetration of the asphalt does not appear to be very vital, so long as the aggregate stability is high, but as this natural stability lowers, the penetration of the asphalt must be lowered to correspond, and with low-stability in the aggregate, greater care must be used in determining the percentage of the bitumen.

Our experience here seems to prove that only reasonable amounts of dust or filler are necessary, with a range between three to five per cent apparently sufficient.

We also seem to have ample proof that the maximum size of the aggregate is relatively unimportant! We have built three and four inch mats of nothing but sand, with no grains larger than 30 mesh, and medium penetration asphalt, yet we developed sufficient stability! However, these grains were very sharp! I have used Kentucky Natural Rock Asphalt, which has very soft bitumen, and very fine grains, 12″ thick in one layer, yet experienced no shoving or corrugation. These grains are sharp as diamonds!

In conclusion, I may say that the one ''outstanding'' feature of our research work is that the ''shape of the aggregate particles'' is by far the most important characteristic of all the materials!

We have not traced these characteristics through all phases of base construction, but everything seems to indicate that the same reasoning will apply.

Design and Construction of Bituminous Surfaces In Alberta

By

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Department of Public Works
Alberta, Canada

Nothing but dust laying oils had been used on Alberta Provincial Highways prior to 1931, but in that year the Public Works Department started to consider bituminous surfacing in a serious way. During that year and the following, some 55 miles of highway were surface treated, using Plant Mix mat of 2 to 2½ inches in thickness, composed of densely graded aggregate and slow curing asphaltic oil of SC-5 type. This surface was laid on old gravel surfaced roads of the horse and buggy type of alignment and design. The roads were first reconditioned by power blade maintainers, and primed with MC-1 asphaltic oil. These surfaces were later sealed, using RC-3 and ½ inch aggregate, from which most of the fines had been eliminated. These surfaces have kept up remarkably well considering the base on which they were laid. They are now however starting to give a considerable amount of trouble in certain sections, which require very energetic maintenance.

The slump hit Alberta in 1932, and no further surfacing was carried out until 1934.

When we started up again, the Department was looking for a cheap form of surfacing, and decided to try the inverted penetration type, otherwise known as Blotter Treatment. Blotter Treatment though practically discarded in other provinces and in the United States, other than for low grade highways, is still being carried on in Alberta on her main highways, with some degree of success; more so now that base course stabilization has been incorporated into the design and construction of bituminous surfacing. Was it not M. E. Gray who stated that "If the subgrade is firm, a very thin ribbon like application of bitumen will serve to carry any amount of traffic"? It is with something like this in mind that we have been trying to evolve and construct highways which will serve as many communities as possible of our sparsely populated

Province. Owing to financial conditions, and distances to be covered, we must have something cheap, and easily and rapidly constructed. We cannot therefore at present go into the higher types of surfacing, and are hoping that with the advent of base course stabilization, that our Blotter Treatment will suffice to adequately carry the traffic, except possibly the more concentrated areas within a radius of some 40 or 50 miles of the cities of Calgary and Edmonton.

Our short experience very quickly taught us that in this type of surfacing the principal consideration must be given, not so much to the actual mat, but to the foundation for such a mat, that is the subgrade and the base. The general standards of our existing highways were far behind the requirements of service and safety, so that our main highways are being relocated and built to a standard which it is hoped will serve the existing and near future traffic requirements. These new graded highways require surfacing as soon as possible, and though good engineering practice indicates that more time should be given between grading and final surfacing, we can only allow the short period of Winter and early Spring months to show up any weaknesses. Consequently it is expected that a certain amount of surface failures are bound to develop, due principally to subgrade and base course conditions. However, we think that we can now state that our first trial in this, to us, new type of construction, that is the base stabilization, covered with a Blotter surface, carried out during the early Summer of 1938, is a success, and a decided step in the right direction.

During 1937 the grading was carried out, and subgrade constructed to the new standards and alignments, between the towns of Macleod and Okotoks, a distance of over 81 miles. Some consideration was given to compaction of fills and grade, and sheeps foot poachers were used for the first time in Alberta. The road was opened to traffic, and three small stretches covering some 4½ miles surfaced with 470 to 675 cubic yards of gravel per mile, where it was considered necessary, to carry the traffic during the spring thaw.

Soil samples along the grade were taken by our Testing Engineer, Mr. L. H. McManus, and with these and samples of the gravel to be used, the stabilized base course was designed.

It was decided to use 2 inch gravel. The specifications called for "Crusher run gravel to be the product of the crusher, of which not more than 20% shall pass one quarter (¼) inch circular opening, and the largest pieces of which will pass through a two (2) inch circular perforation." Fifteen hundred (1,500) cu. yds. of gravel per mile were used, this being the constant factor. The binder materials were taken mostly from the surface of the subgrade, varying according to the requirements as disclosed by the soil and gravel analysis, and the design. In some sections sand, and in others clay, had to be added to the subgrade materials. The P.I. aimed at was between 9 and 15, which is considered too high now.

The following is a sample of instructions given the engineer in the field, by the Testing Engineer:

		Quantities Cu. Yd. Per Mile		
Project	Sta. to Sta.	2″ Crusher Run Gravel	Subgrade Material	Other Material
3 - B - 1	0 - 203	1,500	480	150 sand
	203 - 291	1,500	315	225 clay
	291 - 448	1,500	705	

Clay to be used from Pit at Sta. 502.

Sand to be used from Pit at Macleod Bridge.

This gave the engineer a guide as to the mix to be used in the various sections.

The subgrade was scarified and bladed to get the necessary quantities of subgrade material, and this was bladed into a windrow in the center, and measured. If other material had to be added, this was placed on this windrow, mixed, and windrowed up again. The windrow was then split and a half bladed onto each shoulder of the road. During these above operations the surface was brought as near as possible to the required profile and cross section with crown. Seven hundred and fifty (750) cu. yds. of the 2 inch crusher run gravel was then spread between the windrows of binder material. If the subgrade was very dry, it was dampened before the gravel was spread, to make a bond with the first lift of mixed material. Half of each windrow was then bladed over the spread gravel, and thoroughly mixed by blading and disc harrowing. When sufficiently mixed it was spread, watered and rolled. When thoroughly compacted the second lift of 750 cu. yds. of gravel was applied, and the same process of mixing and compaction carried out. More care to the mixing, watering and rolling was given to the second application than to the first. Compaction was helped by arranging the work so that the gravel trucks had to haul over the work while the process of stabilization was being carried out. Final shaping of the surface was given during the last stages of compaction.

The above appears to be somewhat of a complicated operation, but once the men and machinery were organized, and everyone realized what was wanted, the work went ahead very smoothly, and an average of 1½ miles was completed for each day worked. The machinery used consisted of 2 power blade graders, with scarifying equipment working ahead, preparing the grade, windrowing the binding material, etc., and swinging back if required to help with the mixing. An R.D. 7 Cat., with a 12 foot blade grader, a 30 H.P. Cat., and disc harrows did the mixing, and a power blade grader following up doing the final finishing. Two 10 ton, 3 wheeled rollers and one pneumatic roller did the rolling, and the water was supplied by tanks and distributors from local sloughs and creeks. The cost of this work exclusive of gravel, ran to about $400.00 per mile.

Thorough mixing by blading and harrowing was required, but too much was found to be detrimental, in that it brought too large a percentage of the coarse gravel to the surface. Water used varied according to the weather and soil conditions, the quantities ranging all the way from 10,000 to 50,000 gallons per mile. Besides the trucks hauling through during construction, traffic was allowed to use the road throughout the operations until Blotter surfaced, a period varying from one to four weeks. This helped to show up weak spots in the base and subgrade, which were repaired before surfacing commenced. This year we have done some stabilization work, keeping the traffic off until after surfacing. This does not appear to be a good policy, as some weak places have developed which might have shown up under traffic, and repaired previous to being surfaced. In the 81 miles of work described above, very few breaks have shown up since the surfacing was completed, over a year ago.

As soon as stabilization was completed Blotter Treatment started on June 25th. This consisted of first brooming the surfacing clean of dust and loose material with rotary brooms, then priming with MC-1 at approximately ¼ gallon per sq. yd. After allowing the primer to become completely absorbed into the base, the binder oil was applied at the rate of approximately ⅓ gallon per sq. yd. This was immediately covered by aggregate at the rate of 300 to 325 cu. yds. per mile, 22 ft. wide, which was composed of one inch of crushed gravel as near as possible to the following specifications:

Passing a 1 inch circular opening	100%
Passing a ¾ inch circular opening	90 - 100%
Passing a ⅜ inch circular opening	20 - 50%
Passing a ¼ inch circular opening	0 - 10%

The aggregate weighed about 2,600 lbs. per cu. yd., and gradations varied according to the pits used. Some SC-2 was used as primer, but the general opinion here is that MC-1 makes the better priming oil.

MC-3, 4 and SC-4 and 5 were used for binder oils, with the result that MC-4 and SC-5 are favored as being the most satisfactory for this work, SC-5 being used for aggregates with slightly higher percentages of fines.

The aggregate was applied with the use of Buckeye spreader boxes. We used to windrow the aggregate and blade it over the oil. This method is faster, and may give a smoother riding surface, but we do not consider it as satisfactory as using spreader boxes. Blading caused segregation of the coarse particles from the fines, giving a denser graded mat near the windrows, and a more open mat as the aggregate was moved further and further over the road.

After application of the aggregate, the surface was thoroughly rolled with pneumatic rollers, and finished with a 6 ton three wheel roller. Traffic speed was held down for a day or two, until the surface set, and picking up stopped. Rotary brooms were kept handy to brush in loose aggregate covering sections which showed signs of bleeding, to allow any excess bitumen to blot up as much aggregate as possible.

Other equipment used consisted of two 850 gallon pressure distributors, a 1,700 gallon booster tank, 6 to 8, 4 cu. yd. end dump trucks, with special hitches for spreader boxes; two 10 ft. Buckeye spreaders, and a Barber Greene for loading the aggregate from stock piles. A steam boiler mounted on wheels was used for heating the oil in the tank cars at the sidings.

Work was carried on in two shifts, the average run per working day of 16 hours being a little better than two miles.

The cost including oil and aggregate averaged $1,850 per mile. Asphaltic road oil costs us from 10 to 12 cents per gallon, delivered to railway sidings, aggregate $1.75 to $2.00 per cu. yd. in stock piles.

Work was completed by July 22nd, and as stated above, this surface has come through the year in very good shape. We are now sealing it with MC-4, and ¾ inch aggregate.

The following are some conclusions arrived at, drawn from practical experience during the course of construction, some of which were brought out at last year's Conference, during the discussions.

GRAVEL BASE STABILIZATION ROAD MIX

1. Stabilization should be carried out in layers; Sheeps foot packers and pneumatic rollers for actual compaction, and ordinary rollers for finishing.
2. Too much blading segregates mixture and brings coarse materials to top.
3. Job should be open to traffic during construction, to show up weak places.
4. Plant mix gives better control of quantities than road mix, but road mix is cheaper, and satisfactory if properly supervised.
5. P.I. as high as 12 or 15 does not seem detrimental for certain soil conditions.

BITUMINOUS SURFACING BLOTTER TREATMENT

1. Stabilized surfaces require less priming oil per sq. yd. Penetration slightly less than for unstabilized surfaces.
2. Very slightly damp surfaces absorb priming oil quicker and better than dry surfaces.
3. Wet or damp aggregate tends to make the oil flush up through quickly. Appears to blot up more aggregate. (Note: One of our best sealing jobs was done with the use of ½ inch washed wet aggregate).
4. Bleeding is not detrimental, provided promptly covered, by brooming over traffic dislodged aggregate, or the addition of fresh aggregate.
5. Seems preferable to err on the side of too much bitumen, as this can be easily rectified by the addition of aggregate.

The above outlines only the most important points of the construction work described, and the opinions expressed, and conclusions drawn, are not necessarily those of any other than the writer.

Design and Construction of Bituminous Surfaces
A Brief Report on Current Practice in Oregon

By

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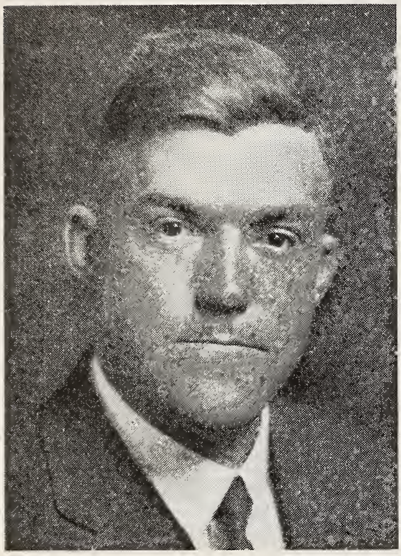
Division Engineer
Oregon State Highway Commission



BITUMINOUS PLANT MIX PAVEMENT

Oregon has 700 miles of two course, five-inch pavement, but has not built much of this type since 1930. This type has been used to fill gaps in similar pavement and has been placed on high fills where settlement was anticipated. We have widened and resurfaced many miles of pavement with bituminous concrete, securing a smooth riding surface and a non-skid, non-light-reflecting texture through the use of a wearing surface built of an open graded mix and cutback binder. This mix has also been successfully used for resurfacing city streets on highway routes.

On Sandy Boulevard in Portland, the abandoned street car track area is being cleaned out and repaved with a concrete base. The street is being widened to sixty feet and resurfaced with a cutback wearing surface for a distance of three and a half miles. The cutback wearing surface one inch thick has crushed aggregate ½″ to dust with only about 15% under 10 mesh and 4½% of RC-4. A similar job performed last year has proven quite successful.

Eight state portable paving plants maintain all bituminous pavement and bituminous macadam surfaces in a constant safe, smooth condition. Fifty thousand tons of hot, asphaltic concrete patching mixture will be placed this year. Some of the pavement is over twenty years old and is in excellent condition, except that most of it is now too narrow and the curves are too sharp for fast traffic. The patching mix used is ¾″ to dust with 20% passing the 10 mesh sieve. Four per cent passes the 200 mesh sieve and 5% of "E" Grade 101-120 penetration asphalt is used.

When any of the older pavements become slick, they are made non-skid by the application of .3 gal. of 121 to 150 penetration asphalt, covered with ¾″ - ½″ and ½″ - ¼″ aggregate spread and rolled. The cost averages about $900 a mile for a width of twenty feet.

BITUMINOUS MACADAM

Oregon has 1,100 miles of penetration macadam two to three inches thick. The first of this type was placed sixteen years ago by Mr. Baldock, who is now State Highway Engineer, and this section is still in use. We have had entirely satisfactory service from bituminous macadam when placed on an adequate keyed macadam base.

A typical heavy section on the Columbia River Highway near Hood River, to be placed this month, has the following specification:

Width—22 feet. **Thickness—3 inches.**

Binder Course:	
Spread RC-3 asphalt	0.4 gal. per sq. yd.
Spread ¾″ - ½″ aggregate	0.01 cu. yd. per sq. yd.
First Course:	
Spread 2½″ - 1¼″ aggregate	0.6 cu. yd. per sq. yd.
Second Course:	
Spread 121 - 150 penetration asphalt.	1.0 gal. per sq. yd.
Spread 1¼″ - ¾″ aggregate	0.02 cu. yd. per sq. yd.
Third Course:	
Spread 121 - 150 penetration asphalt.	0.375 gal. per sq. yd.
Spread ¾″ - ½″ aggregate	.009 cu. yd. per sq. yd.
Spread ½″ - ¼″ aggregate	.003 cu. yd. per sq. yd.
1st Seal Coat:	
Spread 121 - 150 penetration asphalt.	0.35 gal. per sq. yd.
Spread ½″ - ¼″ aggregate	0.006 cu. yd. per sq. yd.
Spread ¼″ - 0 aggregate	0.003 cu. yd. per sq. yd.
2nd Seal Coat:	
Spread RC-3 asphalt	0.25 gal. per sq. yd.
Spread ½″ - ¼″ aggregate	0.006 cu. yd. per sq. yd.
Spread ¼″ - 0 aggregate	0.003 cu. yd. per sq. yd.
Total Quantities:	
''E'' Grade asphalt 121 - 150 pen.	0.725 gal. per sq. yd.
RC-3 asphalt	0.65 gal. per sq. yd.
Aggregate	0.121 cu. yd. per sq. yd.

Each course shall be bladed and thoroughly rolled. Traffic shall use the surface not less than three days between the first and second seal. Temperature for ''E'' grade asphalt shall be 400° and for RC-3 200°. The cost will be $0.73 per square yard.

Heavy hauling on inadequate base may cause failure on this type of surface, but a bituminous macadam will not corrugate or become slick and it is a smooth-riding surface.

OILED SURFACING

Oregon has 2,400 miles of oiled surfacing. Some of this is on stage construction to a higher type. Thickness varies from ½″ to 1⅛″. The aggregate is crushed rock or crushed gravel and the asphalt is SC-6 or tar, a by-product of gas production in Portland. A light single shot specification would be: Broom surface, spread SC-6 at 0.25 gal. per sq. yd. and cover with 0.006 cu. yd. of ¾″ - ½″ and 0.004 cu. yd. of ½″ - ¼″ aggregate and roll. The cost is $0.07 per square yard.

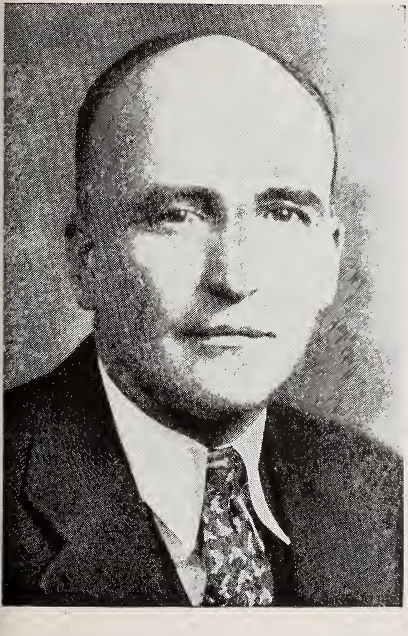
We have used some SC-2 this summer for light oiling work, covering it with material previously windrowed from the surface. In no case is traffic permitted to pass through uncovered oil.

Five state oil crews with retorts, transports, distributors, trucks and rollers operate during the summer. These crews perform reoiling, sealing, and non-skidding work, but practically all new work is handled by contract.

The Use of Rapid Curing Cutbacks in Graded Aggregate Mixes

By

T. V. FAHNESTOCK
Bituminous Engineer
North Carolina State Highway and Public Works Commission



The use of rapid curing cut back asphalt in highway construction in the South Atlantic States has been a development of the last fifteen years and the use of this material has been an important factor in bringing the low cost road to its present status of economic justification and service dependability.

During the middle twenties, the need for all-weather surfacing on secondary roads in the southeast became increasingly apparent and this need led to a number of experiments involving the use of various types of bituminous materials. From these experiments came the bituminous surface treatments on sand clay, and traffic bound macadam bases, now so widely used throughout this area, and also grew the fundamental principles leading up to the design of the graded aggregate mixes using rapid curing cutbacks.

Experiments with slow and medium curing liquid asphalts in mixes did not prove satisfactory. This was largely due to the fact that rain occurs frequently during the construction season, causing extreme difficulty in curing the mix.

Even from the first the rapid curing cut backs gave encouraging results and these materials are now used extensively in the construction of bases, for widening, patching, and for resurfacing.

The specifications for rapid curing cut back asphalts in use by the North Carolina State Highway Commission are shown on the attached sheet. These specifications differ from those proposed by the Asphalt Institute in that requirements are included for the base asphalt and naphtha and also, the viscosity ranges for each grade are divided so that viscosity may be controlled within closer limits. This latter feature is considered desirable due to wide temperature variations encountered in different sections of the State.

In designing mixes using rapid curing cut backs, wide limits in the grading of aggregates are permissible, however, the minus 200 material should be limited to 10 per cent or less for road mix methods, although considerably more fine material can be used where the mixing is done in some type of plant.

In preparing the mixture, sufficient cut back asphalt is used to give a bitumen content of from 3.5 to 6 per cent by weight, depending on the grading of the aggregate and the use for which the material is intended. The mixing is done by harrows and blades in place on the road or in a plant either traveling or stationary. Improvements made in mixing plants, especially the traveling types, in the last few years has lowered the cost of the mixing operation and has greatly increased the use of the rapid curing materials.

The great number of purposes for which the graded aggregate mixes can be used is just now being fully recognized by engineers. Where local sand is available, an excellent pavement can be constructed at a very reasonable cost. These sand bituminous pavements require from 1 to 3 per cent less bitumen content than the hot mix sand asphalt or sheet types. A seal coat is not necessary as these pavements are practically impervious and have been used without sealing for periods up to five or six years. However, one of the most important advantages of the rapid curing cut back asphalt and sand mixtures is the fact that it furnishes a sand paper surface texture which tests have shown to be more non-skid than most other pavements.

These sand mixes are also being successfully used to widen existing pavements, for the superelevation of curves and for resurfacing all types of pavements as well as for patching material. This latter use has become of great importance as practically all of the patching in North Carolina is with premixed material, consisting of either sand and cut back asphalt or a combination of sand with crushed stone or gravel and cut back. The sand mixes are excellent for patching as the mixture can be prepared during favorable weather and stockpiled for later use. This material will remain workable for long periods when kept in a stockpile.

In preparing mixes with rapid curing cut back, weather conditions offer the most serious problem to be overcome. It is an accepted fact that the aggregate should be dry at the time of mixing and it is also true that rain on the mix before a considerable portion of the solvent has evaporated, delays the curing. It is toward solving these difficulties that we have been directing our efforts. We have found that in our sand mixes the cut back can be mixed with the sand when the moisture content of the sand is as high as 8 per cent. This will produce a mixture having a gray appearance and when examined under a microscope, the asphalt can be observed in globules in the mix and the sand grains appear to be coated with moisture and solvent. When the wet mix is dried, the sand grains become coated with bitumen. As the moisture film on the grains contract, apparently the asphalt is drawn around the grains and the mix becomes uniform in color and homogeneous. This same condition is noticeable also when rain occurs on a dry mix before the solvent has evaporated sufficiently to make the coating of bitumen on the grains

North Carolina State Highway and Public Works Commission
Raleigh, N. C.

SPECIFICATIONS FOR RAPID CURING CUT BACK ASPHALTS

This material shall be made from asphalt base and naphtha solvent, so proportioned and mixed that the finished product shall be homogeneous and conform to the following requirements:

Grades	RC-1-A	RC-1-B	RC-2-A	RC-2-B	RC-3	RC-4-A	RC-4-B
Flash Point (Open Tag) ° F.	80+	80+	80+	80+	80+	80+	80+
Furol Viscosity at 122° F.	80–120	120–160	200–250	250–400			
Furol Viscosity at 140° F.					275–400	700–1000	1000–1400
Distillation, Per Cent by Volume:							
Total Distillate to 374° F.	5+	5+					
Total Distillate to 437° F.	12+	12+	10+	10+	3+	0.5+	0.5+
Total Distillate to 600° F.	25+	25+	20+	20+	14+	7+	7+
Total Distillate to 680° F.	40–	40–	35–	35–	30–	25–	25–
Tests on Residue from Distillation:							
Penetration, 77° F. (100 g. 5 Sec.)	60–120	60–120	60–120	60–120	60–120	60–120	60–120
Ductility at 77° F.	60+	60+	60+	60+	60+	60+	60+
Soluble in Carbon Disulph., Per Cent	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+

Asphalt Base: The asphalt base shall be a straight run product, homogeneous, free from moisture, and shall conform to the following requirements:

Specific Gravity at 77° F. — Not less than 1.00.
Flash Point — Not less than 347° F.
Penetration at 77° F. (100 g. 5 Sec.) — 85 to 100.
Loss on Heating (50 g. 5 hrs. at 325° F.) — Not more than 1%.
Penetration of Residue at 77° F. (100 g. 5 sec.) — Not less than 60% of original penetration.
Ductility at 77° F. — Not less than 100 CMS.
Total Bitumen Soluble in CS_2 — Not less than 99.5%.

Naphtha Solvent: The naphtha solvent shall be exclusively a petroleum product and shall conform to the following requirements:

Total Distillate to 200° F. — 0 to 5%.
Total Distillate to 284° F. — 10 to 30%.
Total Distillate to 392° F. — Not less than 92%.
End Point — Not more than 410° F.

water repellent. This does not seem to affect the final results obtained with the material. These conditions do not hold true when the mix is composed of aggregate larger than the number 10 sieve as when the bitumen is once stripped from the coarse particles, it is very difficult to recoat them. For this reason it is found that the sand mixes can be constructed under more unfavorable weather conditions than the mixes containing coarse aggregate. Even recognizing this, rain during the mixing or curing period causes delays which increase the cost of the work and often makes it very difficult to obtain a smooth surface finish.

Experiments are now being conducted in North Carolina to try to obtain a cut back material which will be even quicker curing than the present rapid curing products. From laboratory tests and trial mixes, a tentative specification has been prepared as follows:

TENTATIVE SPECIFICATION FOR EXTRA RAPID CURING RC-3

Flash Point (T. O. C.) Degrees F.	80+
S. F. Viscosity at 140° F.	275 - 375
Distillation per cent by volume:	
Total Distillate to 374° F.	5.00+
Total Distillate to 437° F.	10.00+
Total Distillate to 600° F.	12.00+
Total Distillate to 680° F.	25.00-
Tests on residue from Distillation:	
Penetration at 77° F.	60 - 120
Ductility at 77° F.	100+
Solubility in CS_2	99.5+
Asphalt Base:	
Penetration at 77° F.	80 - 100
Ductility at 77° F.	100+
Solubility in CS_2	99.5+
Naphtha:	
I.B.P.	100° F. to 130° F.
Per Cent off at 200° F.	20 - 40%
Per Cent off at 300° F.	60 - 90%
End Point, Maximum	380° F.
Recovery, Minimum	96%

This cut back is being used in sand mixes, the mixing being done in a portable pugmill and is being used for widening and patching. No sand drier is used with this mixer. It has been found that when the sand does not contain more than 2 or 3 per cent moisture at the time of mixing, the mix can be spread and rolled immediately and develops enough stability to support traffic, however, when the sand contains more than 3 per cent moisture, the mix is somewhat unstable until this excess moisture is dried out. The mixes prepared with the rapid curing cut back material having the light naphtha, spread and finish without difficulty and it is hoped that the reduction in curing time required will result in a substantial saving in the cost of the work.

Design and Construction of Bituminous Surfaces In New Mexico

By

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The New Mexico State Highway Department is ever in quest of a satisfactory top surface, low in cost, that will preserve the large mileage of gravel roads still existent and at the same time handle adequately the traffic on these roads which is for the most part less than 500 vehicles per day.

Oil-processing by various methods and triple-course inverted penetration asphalt surface treatment of gravel for top course have been used successfully, but the cost of these types has been in excess of $4,000 a mile; the average is around $5,000. The department experimented on important but low-traffic-volume roads with single and double-course inverted penetration asphalt treatment of gravel, resulting in a blacktop mat approximately ¾ inch in thickness, costing from $1,400 to $3,000 a mile, including necessary aggregate material.

Our projects have been completed too recently to record any definite experience concerning durability or maintenance economy, but similar types in adjoining states have been both satisfactory and economical and there is no reason to believe that the same good results will not be obtained in New Mexico under similar conditions.

Essentially the same quantities are involved in both types, the main differences being in the manipulation and grading of the cover material.

The average cost has been $2,070 a mile; this includes necessary cover aggregate, base preparation, rolling, road oil for base treatment and asphalt for cover treatment, and manipulation. It does not include engineering costs, contingent items or the cost of adding base material where necessary. The per mile cost varied from $1,400 on the one job that did not require additional aggregate or cover material, to $2,350 on a 44-mile project where suitable cover material was high in price because of the scarcity of local material.

In placing "single shot," the gravel base is first prepared for the asphaltic surface course by the following methods:

All oversize material is removed from the roadway by hand raking.

Where required binder is placed on the roadway in amounts designated by the engineer.

After placing binder on the base course it is thoroughly harrowed or processed so that the binder is evenly distributed through the surfacing as directed by the Engineer. After harrowing or processing the surface is shaped and dragged.

If the surfacing does not compact, the contractor waters and rolls the base course in order to secure proper compaction. The watering is done with a sprinkler or distributor suitable for the work between such stations and in the amounts designated by the Engineer.

The roller is a self-propelled roller weighing not less than eight (8) tons. Rolling where ordered is commenced at either side and rolling parallel with the centerline of the roadway proceeding gradually toward the center uniformly lapping each preceding track and covering thoroughly the entire surface being rolled with the rear wheels. Rolling is done under the supervision of the Engineer and as ordered by him. After rolling and just prior to placing the asphalt the surface of the base course is checked for accuracy with a straight edge templet ten (10) feet long. If the surface varies from the lower edge of the templet at any place by more than one-fourth (¼) inch it is again trued up and re-rolled and this process is repeated until the surface conforms to the above requirements for accuracy.

Where no additional surfacing material is added to the base course, the contractor brings the present base to true cross section by blading and shaping same, when the existing surfacing material is loose; or by scarifying, blading and shaping same when the existing surfacing material is compacted but rough and uneven. The contractor secures recompaction of base by watering and rolling where these items are necessary. After the base is compacted it is shaped to conform to the accuracy requirements noted above.

The base is then ready to receive the cover or surfacing coat.

The wearing surface consists of a single application of approximately 0.5 to 0.6 gallons per sq. yd., of asphalt, penetrating previously placed broken stone or fine gravel spread at the rate of one cu. yd. to 60 sq. yd., of surface, and covered with broken stone or fine gravel spread at the rate of one cu. yd. of cover material to 50 sq. yd. of pavement. The asphaltic material specified is a distillate, uncracked, of 210 to 250 penetration at 25° C., and 50 minimum penetration at 0° C., ductility of residue 50 cm. at 25° C., flash point 232° C., and melting point 30° C. The gravel or crushed stone cover or aggregate material is graded as follows:

Size of Aggregate	Per Cent
Passing ¾″ screen	100
Passing ½″ screen	25 to 75
Retained on ¼″ screen	90 to 100
Retained on 10-mesh screen	100

When this operation is completed sufficiently far in advance of application of the asphalt, the aggregate is spread on the base at the rate of 1 cu. yd. to approximately 60 sq. yd. of area. The asphalt, heated to between 325 and 375° F., is then applied to the surface at the rate of ½ gallon per sq. yd. The surface is immediately covered with the second application of aggregate, spread at the rate of 1 cu. yd. to each 50 sq. yd. of surface, evenly and accurately distributed. A one-man maintainer is employed to insure an even and smooth surface. The wearing coat is then rolled thoroughly with a 5-ton roller. Depressions or irregular spots developing under rolling are patched and the entire surface is brought to section. Traffic is not permitted to use the surface until cover material has been placed and thorough compaction and a permanent surface have been obtained by rolling and patching where necessary.

Base preparation on the "double shot" surfacing is the same as for the "single shot."

The wearing or surface course, commonly called double bituminous surface treatment, consists of two applications of asphaltic material, each covered with a layer of broken stone or gravel. The asphalt used is the same as that specified for the "single shot" type. The grading of cover material is somewhat different. The first cover coat is graded as follows:

Retained on ⅝″ sieve (square opening)	0%
Retained on ½″ sieve (square opening)	0 - 10%
Retained on ¼″ sieve (square opening)	70 - 100%
Retained on 10 mesh sieve (square opening)	95 - 100%

The second cover coat is crushed to meet the following requirements:

Retained on ⅜″ sieve (square opening)	0%
Retained on ¼″ sieve (square opening)	2 - 20%
Retained on 10 mesh sieve (square opening	70 - 100%
Retained on 20 mesh sieve (square opening)	95 - 100%

Assuming the base course has been thoroughly prepared and compacted and accurate as to section, it is cleaned of all dust and loose material, and a prime coat of asphaltic road oil MC-1 applied by a pressure distributor at the rate of approximately one quarter (¼) gallon per square yard of base to be treated. This oil shall be heated to a temperature of not less than 80° F.

After the stabilizing of oil has been applied it is allowed to penetrate for a minimum period of twenty-four hours without being disturbed or for such additional period of time as may be necessary to permit the drying of the prime coat, which period is determined by the Engineer. Traffic is kept off the treated surface during this time to allow the oil to properly penetrate, and the contractor takes suitable precautions to protect the base course against damage during this interval.

After the prime coat has been allowed to penetrate for a sufficient period the surface of the base is then cleaned of dirt, dust or other deleterious matter by sweeping or other approved methods and asphalt is applied on the clean surface by an approved type of self-propelled pressure distributor so operated as to distribute the material in the quantity specified, evenly and smoothly, under a pressure necessary for proper distribution.

Asphalt is not heated above 400° F. at any time, and is applied at a temperature of not less than 275° F. and not more than 375° F. All asphalt material heated above 400° F. is rejected and asphalt is applied in one application for the full width of the surface treatment unless the width exceeds twenty-two feet, to avoid a center seam or mark.

The surface is then next covered with No. 1 aggregate. This aggregate is spread with a spreading device.

The work is then lightly rolled and the second application of asphalt is made in the manner specified for the first application. The second application is covered with No. 2 aggregate and the entire surface is broomed and thoroughly rolled with an approved self-propelled roller weighing between five and eight tons. This rolling is repeated on two successive working days after the application of the No. 2 aggregate. After the work is completed as specified there is a slight excess of aggregate on the surface. The asphalt and aggregate is applied at the approximate rates indicated on the plans within the limits of the following schedule:

Application	Gals. of Asphalt Per Sq. Yd.		Aggregate Cu. Yds. to Sq. Yds.	
	Min.	Max.	Min.	Max.
First	0.2	0.35	1:100	1:60
Second	0.3	0.4	1:200	1:120

Results on these two types of surface treatment have been gratifying. Both present a very smooth surface. On one experimental project where both methods were employed, it was impossible to distinguish between the single and double treatment by casual inspection.

SECTION IV

Maintenance of Bituminous Surfaces

Chairman

T. R. PERRY, Bituminous Engineer
Iowa State Highway Commission
Ames, Iowa

Committee

R. J. TEMPLETON, District Engineer
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Sheridan, Wyoming

J. D. MacVICAR, Maintenance Engineer
Washington Department of Highways
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C. E. SHUMATE, Asst. Supt. of Maintenance
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West Virginia State Road Commission
Charleston, West Virginia

E. W. MEEKER, State Highway Engineer
South Dakota Highway Commission
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Idaho Department of Public Works
Lewiston, Idaho

LEVI MUIR, Materials Engineer, and
WM. F. HUGHES, Soils Investigator
Utah State Road Commission
Salt Lake City, Utah

E. E. BLOMGREN, District Engineer
Michigan Highway Department
Kalamazoo, Michigan

Report of Committee on Maintenance of Bituminous Surfaces

By

T. R. PERRY
Bituminous Engineer
Iowa State Highway Commission
Ames, Iowa

The individual committee papers have been prepared with the idea of developing the practical side of bituminous maintenance rather than the general or technical angle. The Committee had in mind the man in the field who has to decide what to do and when to do it. Anyone interested in bituminous maintenance can well afford to spend some time digesting these splendid papers. They contain gems of advice polished smooth by the hard stone of experience.

Each committee member was asked to submit the topic or topics he was interested in having developed at this conference. This summary will include these topics in a condensed and impartial presentation and no plea will be made for any particular procedure. The conference should draw its own conclusions based on these latest up to the minute developments in bituminous maintenance and on the subsequent discussion.

This Committee reflects the maintenance practices developed in all sections of the United States and by our good neighbors from Saskatchewan, Canada. The various phases of maintenance work requiring the current attention of maintenance men throughout this vast territory apparently group themselves along the following lines:

1. Surface Repairs.
2. Shoulder Maintenance.
3. Base Repairs.

Before developing these three main topics it might be well to consider the interesting responses that were received to the question, ''What criterion do you use to determine when to patch, seal, remix or reconstruct a bituminous surface?''

Muir and Hughes of Utah feel that in order to make a correct determination it is necessary to have a report showing the following observations:

1. A soil survey showing:
    a. The probable cause, extent and type of failure.
    b. Depth and condition of existing gravel base.
    c. Condition of drainage structures.
    d. Position of water table at all seasons of the year.
    e. Soil conditions of the subgrade.
2. A survey of the alignment giving particular attention to present day and future traffic requirements.
3. A study of the probable salvage value of the present road.
4. A study of the financial condition of the state: whether or not, at the present time, the state is financially able to reconstruct.

Many times it is necessary to make rather extensive repairs using a less costly method than hand patching in order to hold the present surface until such time as major changes can be made.

Sanders of Montana believes that a neat formula could be worked out if the necessary financial support was always available but without such they pro-rate the budget for patching, sealing, mixing or reconstructing as the traffic serviceability of the route demands. On sections of roads built to permanent grade patching by hand or near hand methods are used providing the cost does not exceed $50.00 to $100.00 per mile. Beyond that, the mat is torn up, remixed, relaid and sealed. SC mats in good condition are sealed every two or three years and MC mats every year or two. The trend in late years has been to use the heavier MC and SC bitumens with sand or rock chips.

Templeton of Wyoming states that base and subgrade failures are the cause of over 90% of all surface maintenance in that state. Last year the surface maintenance amounted to approximately 60% of the total maintenance expenditure or approximately $137.00 per mile. It is a general practice to scarify and reprocess with additional SC or MC bitumen mats which are dried out or badly cracked. Patching is an endless job in these cases and the results received from sealing are only temporary at best.

Beckham of South Carolina states that South Carolina has a large mileage of ¾″ to 1″ thick "surface treatments" on sand clay, top soil, or stabilized bases. This surface should be treated whenever the mat thickness has been materially reduced or whenever the asphalt shows the first sign of brittleness. Experience has shown that it is necessary to retreat the surface every five years to enliven and reseal the mat and to retain a smooth non-skid riding surface.

MacVicar of Washington has the following to say: "It is a fact well known to maintenance engineers that every failure of an oiled surface is due to a definite cause, and that before repair can be properly made, the cause must be accurately determined, and the faulty condition removed.

"In general, the repair of bituminous surfaces is something that produces for the Maintenance Engineer, more 'headaches' than perhaps any other type of maintenance.

"Failures are principally due to lack of proper drainage and insufficiency of base."

Meeker of South Dakota suggests:

1. Start patching as soon as breakups appear on the surface.
2. Seal as soon as the bituminous material has worn from the aggregate so that the surface has shown a white appearance. Seal coats in South Dakota are merely a wearing course and not a water repellent necessity.
3. Meeker does not recommend remixing or reconstructing but rather to maintain the surface as it exists and, if necessary, place a 1″ retreat over the old mat.

Worthington of West Virginia gives this criterion to us.

1. Patch immediately any hole or break that develops regardless of weather conditions.
2. Seal when the surface becomes checked with fine cracks, or shows signs of becoming dry with a light gray color from oxidation and evaporation.
3. Reconstruct when the surface has become distorted to such an extent that it is impossible with routine maintenance to maintain a reasonably smooth riding surface.

Blomgren of Michigan contends that when the patching of an asphalt surface becomes so extensive that the cost per year is greater than ⅓ the cost of a surface treatment or if the mat has dried out so that it is cracked and open, a surface treatment or seal coat becomes necessary.

SURFACE REPAIRS

This topic automatically divides itself into several sub-heads. Full development cannot be accorded all of these during the time allotted for this paper. We shall hit only the high places which appear to be requiring the most attention. It is interesting to note the similarity of maintenance procedure used in the different parts of the country.

This heading is broken down into the following sub-heads:

1. Patches and patching.
2. Surface treatments.

Patches and Patching

MacVicar of Washington draws some very definite conclusions on this subject and contends that "It is quite disconcerting, and very conducive to a display of bad temper, to start out on a tour of inspection and find on a particular stretch of your pet oiled surface, that since your last trip over, the patching crew has been out, and left its appearance very much resembling one of Grandmother's crazy patchwork quilts. Patches of every shape imaginable, and of every hue under the sun have been made; also the riding qualities of these patches may leave much to be desired.

There is little excuse for any of those ills just described, and one of the first things that should be impressed on the repair organization is, that patching can be done in such a way as to not create an eyesore."

When it comes to actually apply the patch, careful consideration should be given to the quality, color and sizes of mineral aggregate and the type of bitumen to be used. The mineral aggregate should correspond as nearly as possible with that which was used in the original construction, and if like consideration is given to the type and grade of oil, the patch will in the course of just a short time blend more or less perfectly into the original surface that surrounds it. The riding qualities of the patch can be quite nicely regulated, particularly in cases where pre-mix material is used, by using pre-mix composed of smaller rock for "feathering off" on the edges.

"In this State we are running more and more largely to the use of premix material for the repair of broken surfaces. It is becoming the common practice to mix up and stockpile quite large quantities of premix; generally a sufficient quantity estimated to be enough to carry over for a year. All of our premix is manufactured using for binder a mixture 50% MC-2 and 50% SC-2. Some of this is stored in sheds and some outside in the open, but all is preserved in a sufficiently soft and workable condition as to be readily handled, and with no apparent loss of effectiveness. In the case of outside stockpiles it has been found desirable and very effective to seal the outer surface of the pile with ordinary pipe dip. This practice definitely insures that no deterioration will occur for a year."

Blomgren of Michigan contends that whenever holes or depressions begin to appear in an asphalt or concrete surface they must be repaired at once along the following lines:

1. Clean out the hole thoroughly by brushing and if the hole is damp or wet dry by a gentle torch flame.
2. Apply a thin application of prime and thoroughly coat the sides and bottom of the hole as well as an inch or two beyond the boundary of the patch. In cool weather the prime can be flattened out by use of a gentle torch flame.

3. If the hole is over one inch in depth the premix patching material should be placed in consolidated layers of one inch or less in thickness. The finished patch should be made with a slight turtle back hump where the depth varies from the edge to the center. For example, a patch 2″ deep at the center should be built up about ½″ higher at the center than at the edge and after a couple of days traffic the patch will be compressed flush with the grade of the pavement. Deep patches subjected to heavy traffic may require a subsequent application. It is mandatory to give the patch a final check with a cord or straight edge to insure the evenness of the completed surface.

Worthington of West Virginia contends that, ''The method and material used to patch holes and breaks should be determined by the type of pavement surface involved. For example, the holes in a low class, practically worn out road-mix surface should not be patched with a high class premixed material, whereas on the other hand a high class bituminous concrete surface should not be patched with poured patches. A surface which is inclined to be over-bitumenized should be patched with premixed material in which the bitumen can be controlled, whereas a dry surface should be preferably patched by the penetration method because a little excess bitumen carried over the dry surface will help it. Generally speaking, a patch should not be of a higher class material than the surface which is being patched. Either premix or penetration patches will give service in proportion to the quality of workmanship involved in its placing.''

Worthington is of the opinion that it is more economical as a whole to use poured or penetration patches and supports his contentions by the following facts:

1. The majority of the holes to be patched are from 2″ to 6″ in depth and when premixed material is used there is a tendency to fill the holes, regardless of the depth, with this costly material because of the convenience; whereas, the deeper holes should be filled practically to the finished surface with local knapped or crushed base stone and then sealed in, as a poured patch, using a minimum amount of higher priced aggregate and bituminous material.

2. The majority of the holes are in the older and poorer types of pavement and when holes in such a surface are filled with premixed material there is a tendency for the old pavement to break around the new patch of higher class material and quite frequently the result is two holes, one on either side of the newly filled hole. When these holes are filled with local stone and then sealed with higher class chips and bituminous material this sealing is not confined to the area of the patch alone, but is extended out over the adjoining poorer class surface, thus enlivening the old surface and minimizing deterioration around the new patch.

3. When deeper holes are filled entirely with the small aggregate premixed top course material, there is a tendency for the patch to shove or roll during hot weather. It is true this fault can be eliminated by using large aggregate bottom course material in the holes

first, but this necessitates two kinds of material with the necessity for added transportation and storage facilities. This inconvenience alone discourages its use to the point where the workmen use only the surface or top course material.

4. Only highly skilled patching crews are able to obtain good feather-edge patches with the premixed material.
5. During the late fall, winter and early spring seasons when the majority of the patching is necessary the premixed material is difficult to handle unless it can be obtained directly from the plant in small quantities as needed, which is impractical from the standpoint of the large plant producers.
6. It is never practical to use the premixed type of surfacing or patching material unless it can be taken from the truck and deposited directly where it is to serve its purpose, because of the costly rehandling and manipulation expense, particularly during cool weather. Furthermore, the material being bituminous coated cannot be placed in stock piles along the roads without there being a considerable loss in that part which comes into contact with the ground.

Worthington continues: ''From the above, I hope it will not be construed that I am not in favor of the use of premixed material, because I am really very much in favor of this type of material, but do feel that its use, for the most part, should be confined to the leveling, surfacing, and resurfacing of larger areas, rather than for the patching of small holes and breaks.

There used to be the mistaken idea that the patching of holes could neither be done in inclement weather when the pavement was wet, nor in cold weather in the presence of frost. There was also a tendency on the part of maintenance men to allow holes to accumulate and then patch them all at one time. The idea was that the patching could be done more cheaply in this manner, because of the larger volume of work and materials; but lost sight of was the fact that ''a stitch in time saves nine.'' Many of the holes which in the beginning were so small that a handful of material would have sufficed, developed into large cavities which required many times as much material to fill. Furthermore, the discomfort to the motorist was entirely disregarded.

Always be careful to not over-bituminize a patch—it is better to have to replace a patch than to have a ''fat'' one, because this ''fat'' patch with its bad looks, warm weather bleeding and slipperyness will stay with you indefinitely, furthermore it is a useless waste of material. This warning against too much bitumen should also apply to all leveling courses which are used to bring out depressions or other irregularities. Make the leveling courses lean and thus avoid having an excess of bitumen to bleed through the new surface.''

Dennis of California concurs practically 100% with Mr. Worthington and in addition recommends that cracks in the more rigid type surfaces should be filled with a blended filler made up of 25 to 30% SC-6 and 70 to 75% of 31 to 40 penetration air blown asphalt.

Roadmixed patch materials made with local aggregate are cheaper han either plant mix or penetration types. In California road mixed patch naterial made with local aggregate cost $1.00 to $2.00 per ton in the stock pile.

The average costs of different types of patching per square yard for one inch thickness are as follows:

Road mix using SC-2, SC-4, ROMC	.12 to .25
Plant mix with the same bitumen	.20 to .36
Penetration using Emulsified asphalt	.20 to .25
Penetration using SC-6	.22 to .35

Premix and penetration patches give about the same service under average conditions but in prevailing frost areas the penetration type of patch is likely to ravel and disintegrate if more than one rock in thickness.

Winter Maintenance

Dennis also advises that under severe winter conditions holes and base failures are patched with untreated or local material. These patches are replaced with permanent ones when warm weather comes. Both penetration and premixed types are used. Premix is used almost exclusively where wet weather prevails. Emulsion and screenings may be used even if the surface is damp in those areas of intermittent wet weather.

Templeton of Wyoming records that only a small amount of surface repair work is carried on during winter months. This work consists mostly of skin patching (penetration) with RC-2 cutback and stone chips, and a limited amount of sealing during periods of favorable dry weather. Snow removal is of unquestionable value in the maintenance of bituminous surfaces as snow allowed to melt on the shoulders is liable to saturate and weaken the base and cause serious trouble.

Humphrey of Idaho advises that in localities where sub-zero temperatures and heavy snow or where light snow and wet and cold weather prevail neither penetration nor premix patching is successful. The best patching method consists of tamping aggregate and mineral or soil binder into the holes and then repatch with bituminous material in the spring. In localities having cold weather and a short wet season with low precipitation, heated premix can be used to good advantage provided the holes are dry and free from moisture.

Surface Treatments

Surface treatments in this summary shall include all types of treatments other than patching which are designed to seal, level, recondition or add structural strength to the existing surface. They range from the light penetration seals to the heavy retread machine mixed types of treatment. Both asphalt and Portland Cement concrete pavements will be included.

A number of the papers signal the increasing need of smoothing up old Portland Cement concrete pavements.

Muir and Hughes of Utah have developed a treatment to repair old Portland Cement concrete surfaces by using native rock asphalt from Sunnyside, Utah. The usual procedure is to use about ½″ leveling course and then apply a wearing course of about 1½″ thickness. Certain fundamental rules must be followed to secure satisfactory results.

1. The rock asphalt must be ground fine enough so that all passes a ½″ square hole screen.
2. The minimum bitumen content shall be 10%.
3. Rock asphalt shall not be laid when the air temperature is below 70 degrees F. or when the rock asphalt contains more than 5% moisture.
4. All bituminous patches and excess expansion joints must be removed before the asphalt is applied.
5. Rolling is a very particular operation and must be done at the right time. After the initial rolling, any additional compaction should be secured by rolling during the cool of the early morning hours except that the final surface rolling should be carried on during the hot part of the day.
6. During compaction the road must be kept closed to traffic.

The average cost of this treatment is about sixty-five cents per square yard.

A mixed-in-place method is also developed using an open graded type of crushed rock aggregate ranging in size from ¾″ to ⅜″ and ⅜ gallon per square yard of RC-4 bitumen. After this mix is laid and rolled 10 lbs. of ⅜″ to ⅛″ chips are applied. The road may then be opened to traffic. Later a seal treatment of ⅜ gallon per square yard of RC-4 and 25 to 40 lbs. of ⅜″ to ⅛″ chips are applied. The cost of this type of treatment is about thirty cents per square yard.

Both types of treatment proved satisfactory.

Shumate of Colorado has, for the past few years, been resurfacing sections of old Portland Cement concrete pavements with a premixed ¾″ maximum sized aggregate in an open graded mix and MC binder. Excellent results have also been obtained by using a close graded mix with the same maximum sized aggregate and SC-3 binder. Several sections have been down for a number of years under 4,000 to 5,000 vehicles per day and still are giving excellent service.

Dennis of California finds that penetration armour coats, retreads, road mixed or plant mixed heavy treatments give good results when used to cover old Portland Cement concrete pavements where there is no movement of the segments. Where there is curling or tilting of segments of the concrete pavement, cracks will be transmitted through these surfaces from the old base below. To secure the most permanent results a cushion course of untreated gravel or crusher-run stone should be placed and then covered with either a penetration, road mix or plant mix wearing surface. This cushion permits vehicle loads to be distributed over a greater area and any movement of the original pavement is absorbed. Traffic loadings and the condition of the pavement and the subgrade determine the thickness of the cushion. Usually the minimum thickness is four to six inches and twelve inches thick where alkali or ground water is responsible for the base failure.

Seal Treatments

Touching next upon seal coat treatments we find that this phase of maintenance holds the attention everywhere.

Blomgren of Michigan includes in his paper tabulations for a variety of treatments for distributor applications of bituminous materials. The tabulations are for the field man to use in choosing the proper treatment and gives him the steps or operations in order that they shall be performed together with the proper quantities and combination of materials to use. Blomgren concludes that the average maintenance superintendent, with his experience, and the aid of these tables and specifications should be able to do a satisfactory job of surface treating on asphalt or concrete pavement.

These tables cover the following treatments:

Class A-1 — Double seal on gravel base.
Class A-2 — Double seal on stabilized gravel.
Class A-3 — Double seal on waterbound macadam.
Class A-4 — Single seal on bituminous retread.
Class A-4b — Double seal on bituminous retread.
Class A-5 — Single seal on oil aggregate.
Class A-5a — Single seal on sheet asphalt.
Class A-6b — Double seal on sheet asphalt.
Class A-7a — Single seal with bond coat on concrete or brick.
Class A-7b — Single seal on concrete or brick.

Templeton of Wyoming uses two types of penetration seal treatments.

1. .12 gallon RC-2 and 5 to 10 lbs. of sand per square yard.

2. .25 to .3 gallon RC-4 and 20 lbs. of stone chips per square yard.

Dennis of California uses two types of seal treatments on bituminous surfaces.

1. Non-skid Treatment which is confined to areas that have become slippery and dangerous to traffic because the excess asphalt has flushed to the surface under the action of weather and traffic. Excellent results have been secured by using rock asphalt in a layer not over 1½″ thick. The usual treatment is 1/6 to 1/7 gallon SC-6 penetration type emulsified asphalt and 30 to 35 lbs. per square yard of clean extra hard screenings ½″ to ⅜″ in size.

2. Seal Treatment which is put on to prevent the entrance of water into the base and eliminate surface ravel. This treatment is constructed similar to the non-skid treatment except that smaller sized screenings are used.

Softer asphalts such as ROMC, SC, etc. may be used in the Pacific Coastal areas without bleeding. If high summer and mild winter temperatures prevail the harder MC and RC liquid asphalts are satisfactory. Where the winter temperatures are severe, use the more ductile ROMC and SC-3, SC-4 and SC-6 bitumens.

SC-2 oils are not entirely satisfactory in winter areas and SC-6 will bleed under high summer temperatures. Emulsion seals properly applied give good service regardless of weather and temperature conditions.

Beckham of South Carolina develops a 50 lb. treatment consisting of .6 gallon of RC bitumen and 50 lbs. of graded ⅝″ to 100 mesh sieve aggregate applied as two separate graduations which have a normal size of ⅝″ to 100 and ⅜″ to 100. The tendency is toward developing one graduation that can be obtained locally. The treatment is constructed by the Road Mix method using a multiple blade maintainer. Ten trucks, one distributor and one finishing machine can apply this treatment at the rate of about 3,000 square yard per hour. The actual cost of this treatment in place is about twelve cents per square yard. This treatment should be applied once every five years.

South Carolina's experience indicates that inverted penetration types of treatments do not materially improve the smoothness and riding qualities of the old roads. The above mentioned mixed-in-place treatment corrects these faults and in addition reseals and adds to the thickness of the old mat.

Worthington of West Virginia believes that seal treatments on the surface are necessary for the following reasons:

1. To prevent the intrusion of surface water with its resultant tendency to start disintegration.
2. To prevent the intrusion of air with its tendency to accelerate oxidation and evaporation, which causes the bituminous material to become brittle and break up.
3. To liven up and make more flexible the existing surface by the penetration of the new oil into the old dry and hard surface.
4. To cover the existing surface with a fine grained sandpaper like finish composed of a finely graded aggregate, which furnishes the skid resistance which is so desirable.
5. To eliminate the surface voids which hold the surface water.

The use of tar or asphalt emulsion is recommended for their better penetrating quality and they offer better skid resistance. The rate of application of bitumen is .2 to .25 gallon per square yard and eight to ten lbs. of ⅜″ aggregate. Periodic application of this light seal will almost entirely eliminate patching.

Three other types of seal treatments have been developed, namely:

1-A. 25 lb. treatment consisting of 20 lbs. of # 12 aggregate, 5 lbs. of # 12a aggregate and .35 to .45 gallon per square yard of RT-7, RT-8, RT-9, EA-3, RC-2 or RC-3. This seal treatment is to be used if finances permit instead of the eight to ten pound treatment.

2-A. 55 lb. treatment consisting of .55 to .65 gallon per square yard RT-7, RT-8, RT-9, RT-10, RC-3 or RC-4 and 40 lbs. of #10 aggregate, 5 to 7½ lbs. of # 12 aggregate and 5 to 7½ lbs. of # 12a aggregate. This type and weight of seal to be used when the surface is dry, open, cracked, somewhat ravelled or uneven and also when additional structural strength is needed.

3-A. 90 lb. treatment consisting of .8 to .95 gallon per square yard of the same bitumen as the 55 lb. treatment and 70 lbs. of #7a aggregate, 13 lbs. of #12 aggregate and 7 lbs. of #12a aggregate. This treatment to be used to build up additional structural strength on a surface which is dry, open, cracked, ravelled and uneven.

The normal size of the above mentioned aggregates are roughly within the following limits:

No. 7a — ¾″ to No. 4.
No. 10 — ½″ to No. 4.
No. 12 — ⅜″ to No. 8.
No. 12a — No. 4 to 100.

The ten pound and twenty-five pound treatments are constructed by the penetration method. The fifty-five pound and the ninety pound treatments are built by a combination of penetration and mixing methods.

Reprocessing Old Mats

Templeton of Wyoming and Sanders of Montana both developed along similar lines the remixing of old bituminous mats which have become rough, cracked, corrugated, dry, ravelled or oxidized. These old mats are torn up, pulverized, mixed with additional bitumen and additional aggregate if necessary and relaid.

Dennis of California believes that, "The advisability of salvaging and reprocessing the existing oil mat during base reinforcing operations depends on:

1. Value of the existing mat.
2. The desirability of retaining the mat as a membrane to cut off the entrance of ground water from below.
3. The retention of the compacted base which might be seriously disturbed by removal of the mat."

Mr. Dennis also sets out the advantages of Plant Mix and Road Mix methods and summarizes the cost of twenty-five construction projects. The Plant mix projects cost more per mile than the blade mix jobs but the maintenance cost per mile is less on the Plant mix jobs.

SHOULDER MAINTENANCE

The concensus of opinion seems to bear out the fact that shoulder construction has not kept pace with the base and mat construction. The result is that many mat and base failures are due to moisture seeping under and through porous or cracked shoulders.

Shoulder treatment is expensive and perhaps should be included as a part of the construction project rather than undertaken as a maintenance duty.

Shumate of Colorado has determined after two years of experimenting that the best way to prevent water from entering under the bituminous mat from the shoulder is to construct an asphaltic membrane adjacent to the pavement. .3 gallon of MC bitumen is applied per square yard over a clean area at least two feet wide adjacent to the mat and allowed to penetrate. One foot of the mat and one foot of the above primed area is given an application of ¼ gallon per square yard of RC-2 and covered with loose gravel.

Last year Colorado placed a premixed mat four feet wide and 2½″ deep, properly rolled and sealed, along the edges of 150 miles of narrow Portland Cement concrete pavement at a cost of approximately $1,500.00 per mile.

Templeton of Wyoming has done considerable shoulder sealing using SC, MC and RC bitumens with satisfactory results. Some mats have been widened to eliminate the shoulders entirely. Broken down edges of bituminous surfaces are invariably due to lack of shoulder support. In order to remedy the condition, Wyoming has had to stabilize the shoulders and flatten the embankment slopes to obtain lateral support.

Dennis of California advises that shoulders on bituminous roads in California are treated primarily for stabilization and the emergency use of traffic. Base saturation is reduced by such treatment even though that is not the chief purpose in oil treating shoulders. Mr. Dennis develops in detail these three methods.

1. Penetration method for earth shoulders consisting of ¼ to ½ gallon per square yard liquid asphalt. The choice of material to use depends on the type of soil encountered. Use SC-1A on fine or lightly bound soils and allow time to penetrate, then cover any free oil with roadside material. An average of three to five cents per gallon will cover this type of work.

2. The Road Mix method is used when the mat obtained by penetration is not sufficient to stand traffic or where, after repeated penetration applications, the mat has become distorted. About one-half of the thickness of the old mat is disced or scarified up and pulverized and mixed with SC-2, SC-3, or SC-4 at the rate of 1/5 to ¾ gallon per square yard. The approximate cost of this work using a multiple blade mixing drag costs approximately two to four cents per square yard. Machine mixing costs approximately three to five cents per square yard for one inch thickness. Road mixing with graders costs from six to ten cents per square yard for one inch thickness.

3. Plant mix is used in heavy traffic areas and costs $4.50 to $6.50 per ton in place. SC-4, ROMC, and MC liquid asphalts are used for binder.

Humphrey of Idaho has found that in Idaho an application of .3 gallon of bitumen covered with sand will protect the shoulders from water or wind erosion. SC-3, RC-3 and RC-4 have all proven satisfactory. The application is usually three feet in width applied so that 1½ feet of the shoulder and 1½ feet of the mat or base are covered.

BASE REPAIRS

Meeker of South Dakota states that South Dakota has had serious trouble along the edges of practically all mats ranging in thickness from 1″ to 3″ that have been in service from two to six years. The sub-soil lends itself readily to capillary action and has a high shrinkage factor. During dry weather cracks open up possibly two inches wide and several feet deep. This condition is prevalent through the state. During wet weather moisture enters these cracks and sets up a condition which causes unstable subgrade conditions. So far, no feasible means have been developed to waterproof the shoulders due to this severe cracking condition.

The repair method developed during the past year is to break up and salvage the outer six feet of the mat three feet on each side and change the texture of the soil by the addition of granular material. This action is secured by sheep-footing the old gravel base into the subgrade material. The old gravel bases are approximately 1,600 cubic yards per mile. After this rolling and compaction, additional stabilized gravel material is added to bring up the base to the bottom of the adjoining mat. The base is then primed and the old reconditioned mat replaced.

The cost of this work on a 3″ mat replacement of six feet width is approximately $1,500.00 per mile.

The old mat is processed at a central point and not on the road surface. The road had to be kept open for traffic.

Templeton of Wyoming advises that in northern Wyoming it was necessary to stabilize the base on many miles of old road. The mat was scarified up and laid to one side of the road. The subgrade was then shaped and gravel with the proper amount of suitable binder was hauled in, spread, watered and thoroughly rolled. The old mat was then processed and laid.

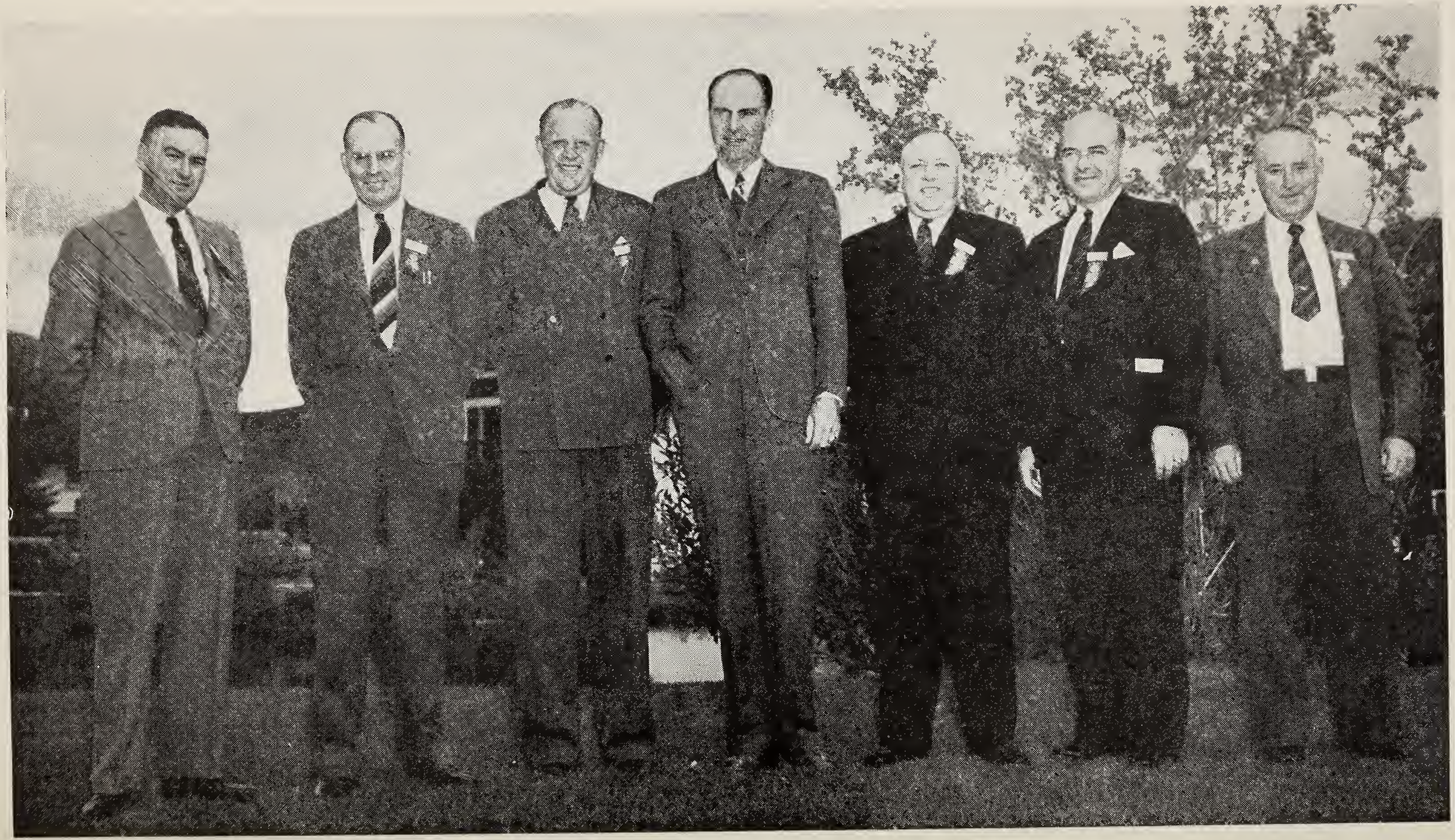
Dennis of California advises that fine aggregate mixing emulsion has been used in stabilizing existing bases and imported materials. When warranted, the old mat has been salvaged but in most cases it has been incorporated with the base and the resulting mixture stabilized. The choice of any method of stabilization should be based on a thorough economic study. Where base thickness is of primary importance and suitable binder and crusher run or gravel base is available, the use of emulsion for stabilization would undoubtedly be uneconomical.

Worthington of West Virginia believes that base repairs can be kept to a minimum if proper preventative measures are employed, most important of which are:

1. Drainage — Water must be kept away from the base and subgrade.
2. Proper side ditching including drains to allow trapped water to drain out through the shoulders.
3. Proper drains at the low points in the grade line.
4. Subgrade drainage to keep the water table lower than the top of the subgrade.

When a section of base does fail, the only solution is to replace the base material and any unstable sub-base with new base material. A good shoulder drain is always necessary where a base failure has been repaired.

Sanders of Montana follows the same line of thought relative to drainage when he concludes that Recurring and Chronic base disturbances in separated and exaggerated cases are disposed of by subgrade drainage with either open joint tile, French drains, porous backfill or perforated pipe. Since moisture is the confounding spirit of the majority of base failures, it is rare when proper subgrade drainage will not provide a permanent cure.



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(Left to Right): H. R. MacKenzie, E. W. Meeker, C. E. Shumate, S. B. Sanders, T. R. Perry, J. G. Schaub, and Levi Muir
(Mr. Templeton and Mr. Humphrey not shown.)

Conference on Maintenance of Bituminous Surfaces

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Mr. Perry: Gentlemen, our good friend, Mr. MacKenzie, came down to this meeting on very short notice, and was unable to prepare a paper. However, he has very kindly consented to say a few words and in a moment I will ask him to take the floor. After Mr. MacKenzie finishes, I have a few slides to show you. These slides were furnished by the various members of the committee, but it is apparent that none of the members furnishing the slides are here to explain them, so I will fill in as the slides are flashed on the screen.

Mr. MacKenzie attended the first Montana conference held at Butte in 1935. We are very glad to have him here again, and I will now ask him to come forward and talk to you. Mr. MacKenzie:

H. R. MacKenzie: Saskatchewan

Mr. Chairman and gentlemen, before saying anything in connection with the subject of Maintenance of Bituminous Surfaced Roads, I feel I must take this opportunity of expressing my deep appreciation of the kindness of Mr. McKinnon and his executive in extending to us in Western Canada an invitation to attend this Conference. I have had the privilege of attending a previous Conference in Butte, Montana and I am very pleased indeed to be back again and to meet some friends whom I had met on previous occasions.

We in Canada, and myself in particular, are deeply indebted to the engineers of the states of Minnesota, Montana and North Dakota for their kindness in giving us the benefit of their extended experience in connection with bituminous surfacing work. These officials have acted as big brothers to me during the past fifteen years, and what little I know of bituminous surfacing work is due almost entirely to the information I have obtained from engineers on this side of the line.

Reverting to the subject, I think Mr. Perry is to be very heartily congratulated on the splendid manner in which he has summarized the various papers on the Maintenance of Bituminous Surfaces.

I know the maintenance of any type of road is not so much knowing what to do or how to do it, but to keep continually doing what you know should be done. It is a matter of persistence, and I would like to say that in driving over the excellent roads in the state of Montana I saw some examples of very careful and efficient maintenance work, where small holes which did not require more than a cupful of material were being patched. You know that is a very tedious process, but it was being done in a very efficient manner.

In this state the feature of maintenance work which I noticed most particularly, Mr. Chairman, was the excellent condition of the center traffic stripe, and the almost uncanny accuracy with which this stripe was located. I wish to congratulate whoever is responsible for that particular phase of the work in the state of Idaho.

The province which I represent is north of the international boundary line abutting Montana and North Dakota. We have a settled area about 400 miles square, over 225,000 miles of road, and approximately 8,000 miles of what you would call state highways. We have only a limited mileage of bituminous surfacing. We are now using plant mix bituminous mat about four inches in thickness, using SC-5 asphalt. We have a large area of heavy clay soil and a very limited amount of gravel; consequently, we are very glad to avail ourselves of the principles of soil stabilization which, by the way, was developed by the United States Bureau of Public Roads. Applying these principles, we are constructing a stabilized gravel subbase. The results are very satisfactory, except that at the present time we haven't sufficient money to build a subbase to the depth that would really be required for a permanent type of construction.

The patching work is carried out by hand, and we endeavor to follow the principle recommended by Mr. Perry in repairing these small breaks as rapidly as they occur.

We have considerable difficulty with shoulder maintenance, which I note is somewhat general, and I would like to be advised, Mr. Chairman, if the stabilization of the shoulder and the adoption of a 4:1 slope will largely eliminate the difficulty which we have with shoulder maintenance.

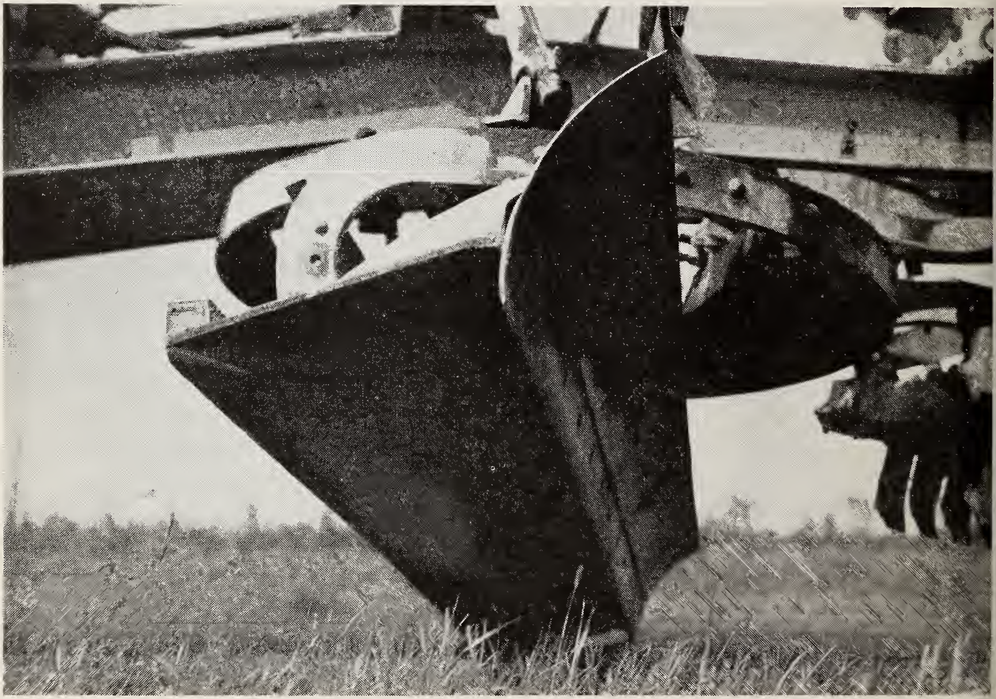
Another feature I am very much interested in is the shrinkage cracks which were reported to occur, I believe, in the state of North Dakota. I was very much discouraged this spring to find that a bituminous mat which we built last year, and which I thought was a pretty satisfactory type of construction, during the extreme hot weather and dry weather of the month of July of this year developed longitudinal cracks almost exactly on the edge of the pavement and sometimes 8 or 10 inches inside the edge of the pavement, which opened to a width of 2½ to 3 inches and extended to a depth of 2 feet. I have discussed this problem with some of the gentlemen here who are more familiar with this work than I am, and consensus of opinion, I believe, is that they are shrinkage cracks.

The subgrade in this particular area is clay which has a plasticity index of at least 50. In some cases the samples required over 100% of water before the material would reach the liquid limit, and you can understand that in material of that nature the volume changes are very great indeed. I would like it if somebody could give me some more information as to the best method of dealing with these very disagreeable cracks which have occurred longitudinally near the edge of this pavement.

I must again express my appreciation of being allowed to be here, and must thank you, Mr. Chairman, for the opportunity of making these remarks.

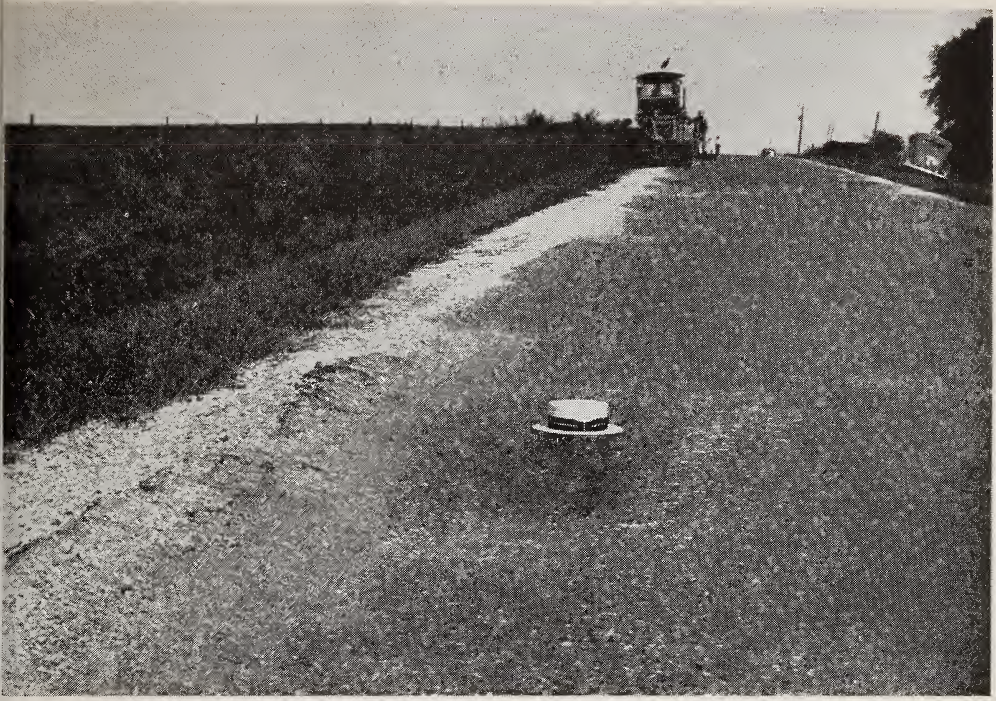
Mr. Perry: Thank you, Mr. MacKenzie, and I want you to know we are all mighty glad to have you here.

Let us have the slides now, please.



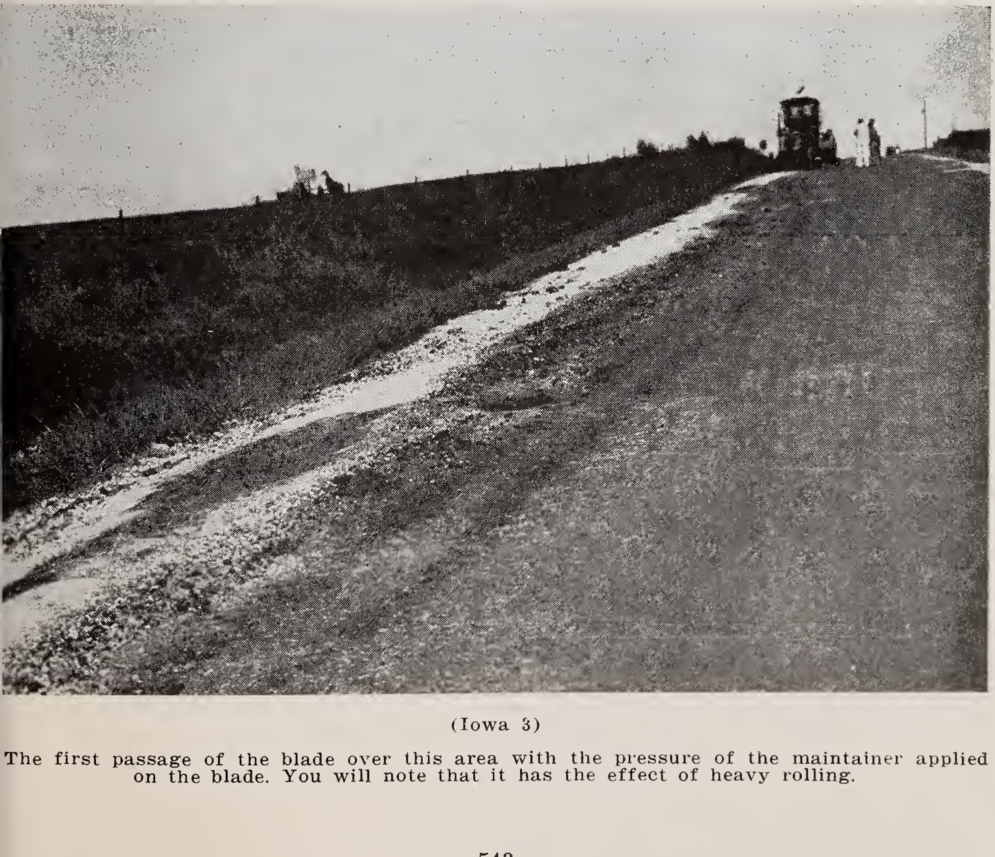
(Iowa 1)

A new blade which was recently developed by one of our foremen and we tried it out rather extensively about 2 weeks before the convention. This blade has a curved surface and is hooked onto the regular maintainer moldboard. The blade is about 2½ inches thick and weighs approximately 900 pounds. The length is 10 ft. for a 12 ft. moldboard. This blade will take the place of a disc and roller when making temporary repairs. We think we can make some very good use of this on some of our older roads.



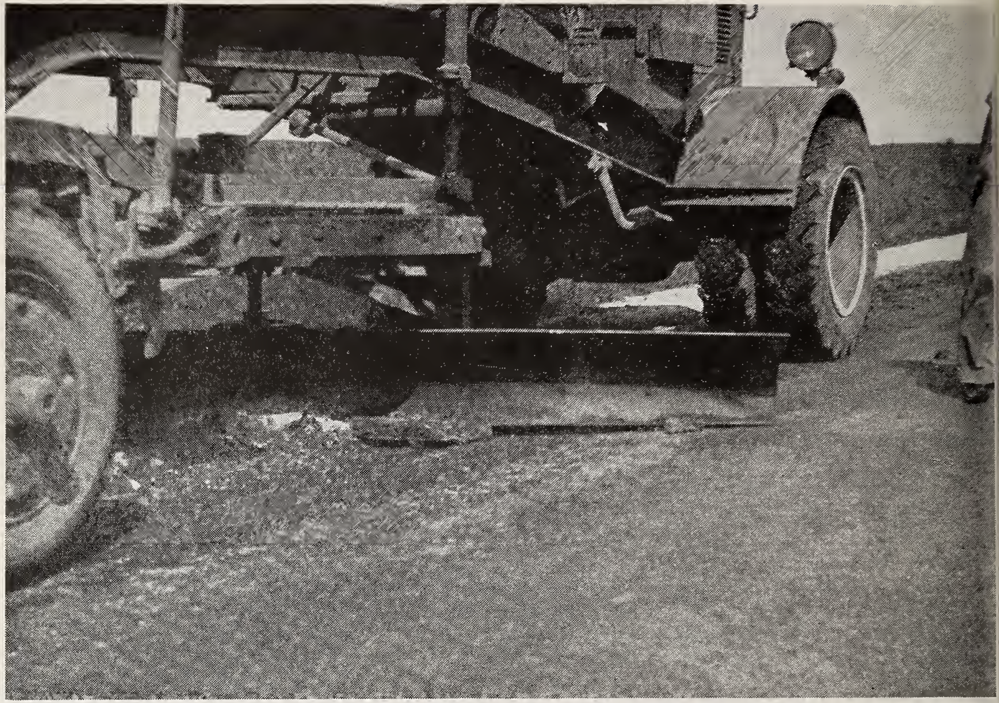
(Iowa 2)

A typical subgrade failure wherein the mat has pushed out toward the shoulder. The hat in the picture represents roughly the depth of the failure; namely, about 7 inches deep. This is the failure that the following pictures show being repaired.



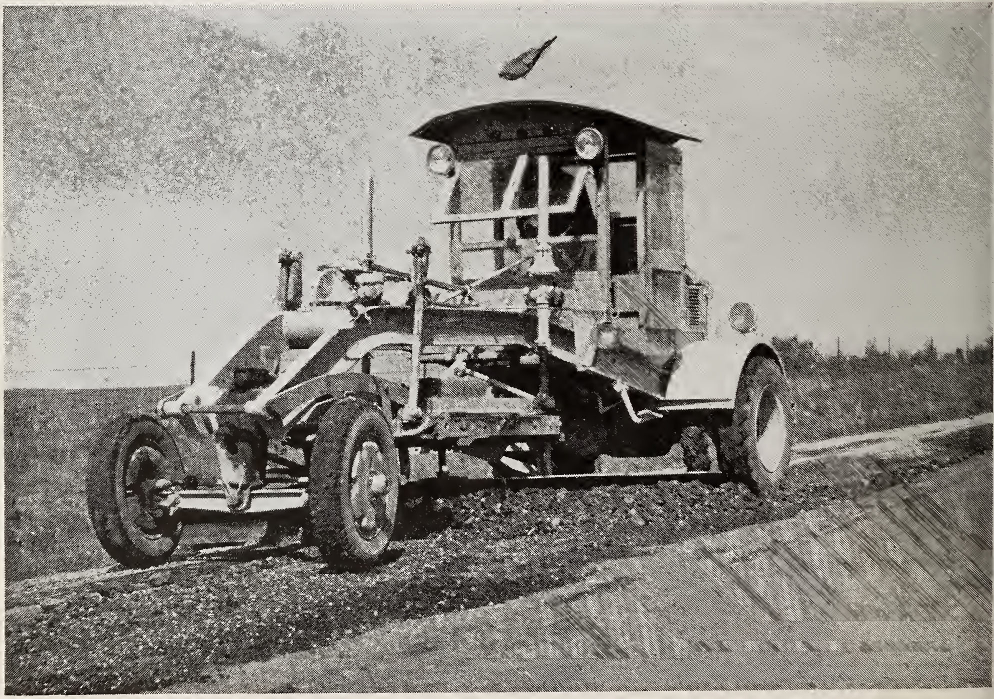
(Iowa 3)

The first passage of the blade over this area with the pressure of the maintainer applied on the blade. You will note that it has the effect of heavy rolling.



(Iowa 4)

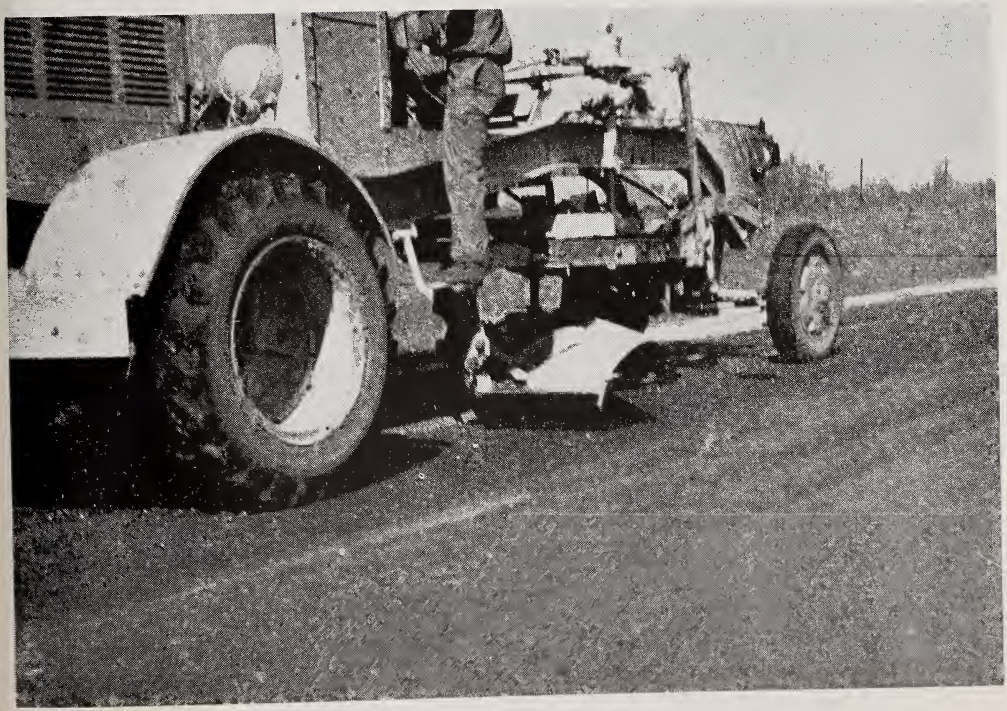
This shows the next step which is to cut off the uneven areas and the picture indicates the size of some of the pieces of mat that were cut off.



(Iowa 5)

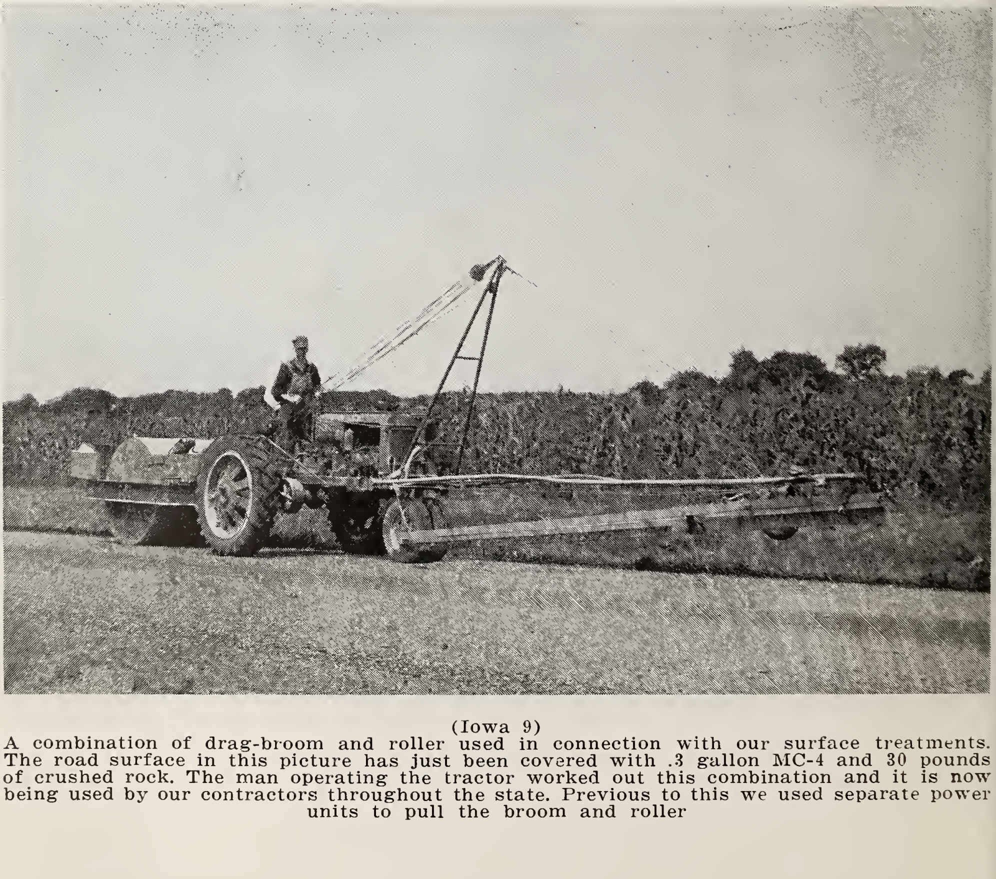
This shows more cutting and trueing up.

(Iowa 6)
The blade used as a pulverizer, and we consider it a very good tool to use for pulverizing the old mats.

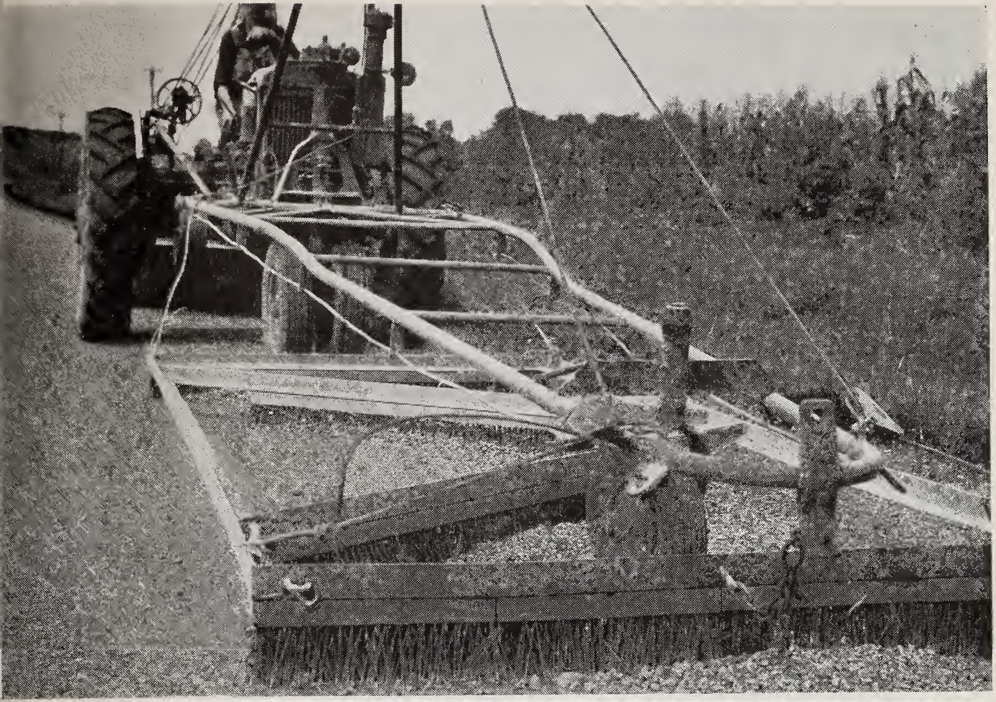


(Iowa 7)
Replacing the old mat material after being reconditioned

(Iowa 8)
The same spot as picture No. 2 after it has been repaired. The only extra equipment that is needed is that it is a good plan to shoot a prime coat over the failed area before starting to work on it.

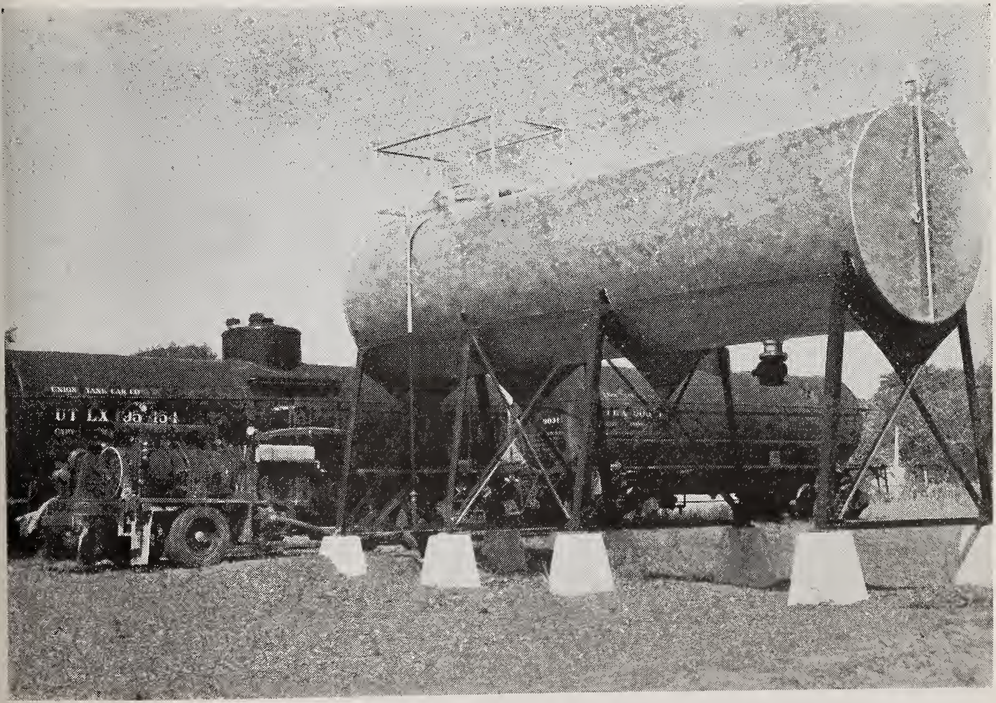


(Iowa 9)
A combination of drag-broom and roller used in connection with our surface treatments. The road surface in this picture has just been covered with .3 gallon MC-4 and 30 pounds of crushed rock. The man operating the tractor worked out this combination and it is now being used by our contractors throughout the state. Previous to this we used separate power units to pull the broom and roller



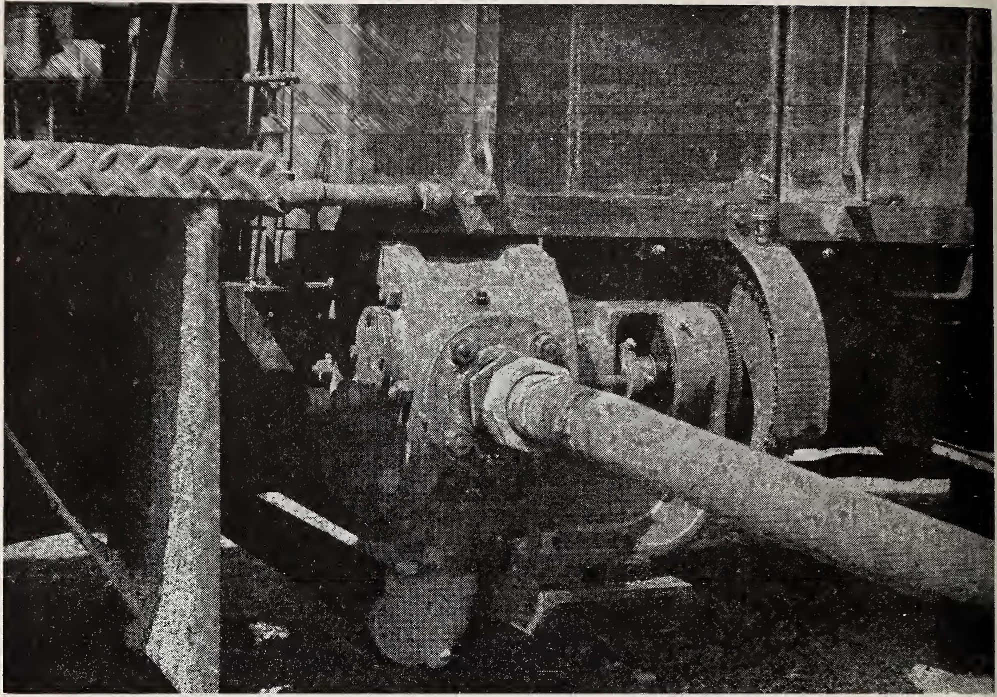
(Iowa 10)

This shows the broom arrangement.

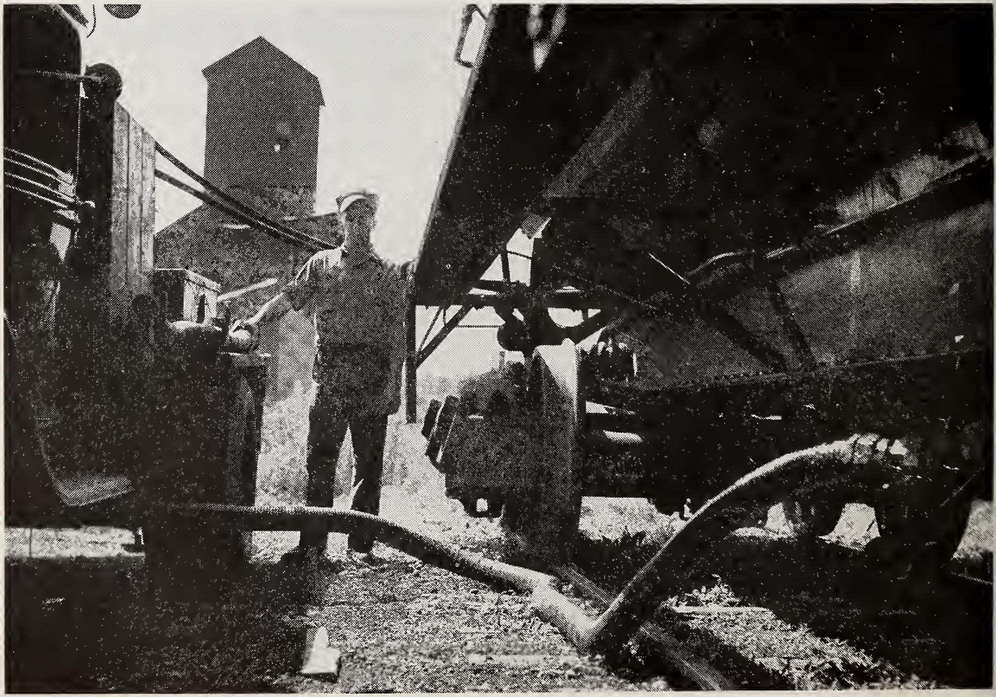


(Iowa 11)

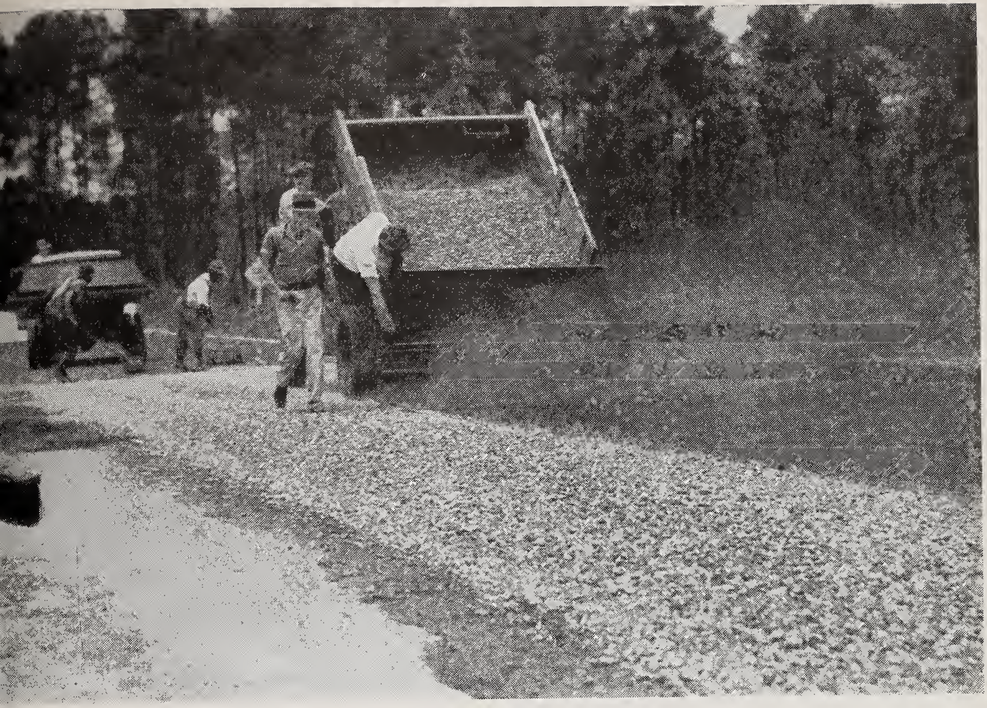
One of our new 11,180 gallon bituminous storage tanks. The unloading unit is a Cleaver-Brooks heater which used to be a trailer type but we mounted it on a Ford truck and equipped the unit with a pump so that one man can heat the car and pump the bitumen into the storage tank. The hose from the pump hooks onto the overflow pipe on the tank and this pipe serves two purposes, both as a loading device and also as an overflow safety proposition. The opposite end of the tank shows an 8 inch valve through which the maintenance forces draw out the bitumen as they need it. There are no heating coils and the unit works just fine except it is rather slow in real cold weather.



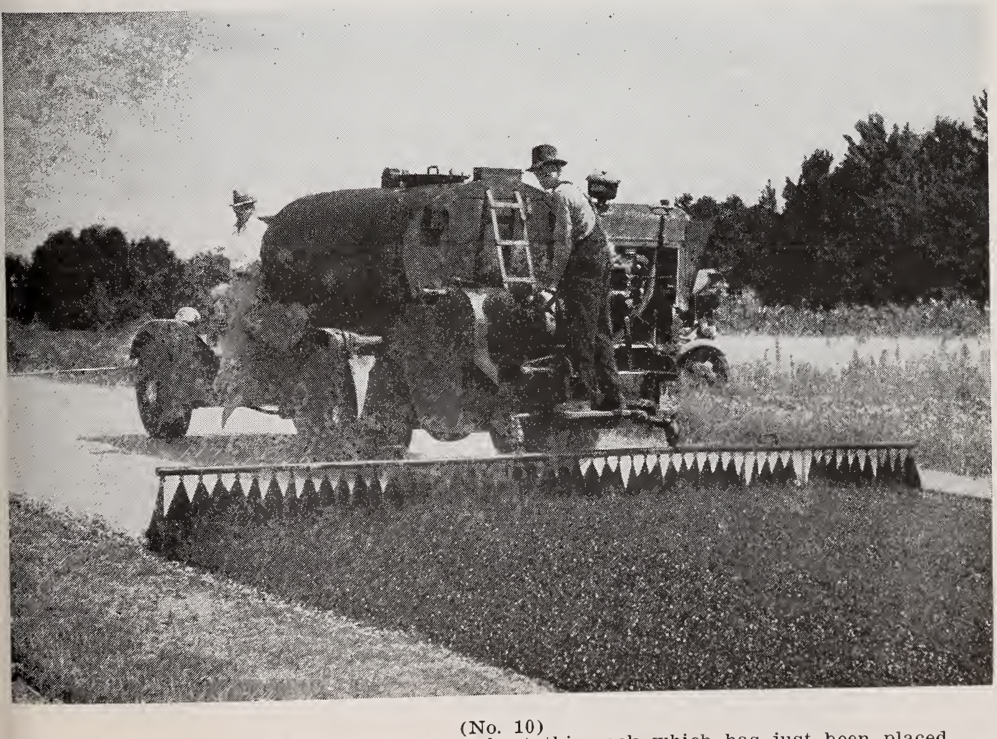
(Iowa 12)
The pump assembly which is a 200 gallon Viking Pump and operated by a chain drive.



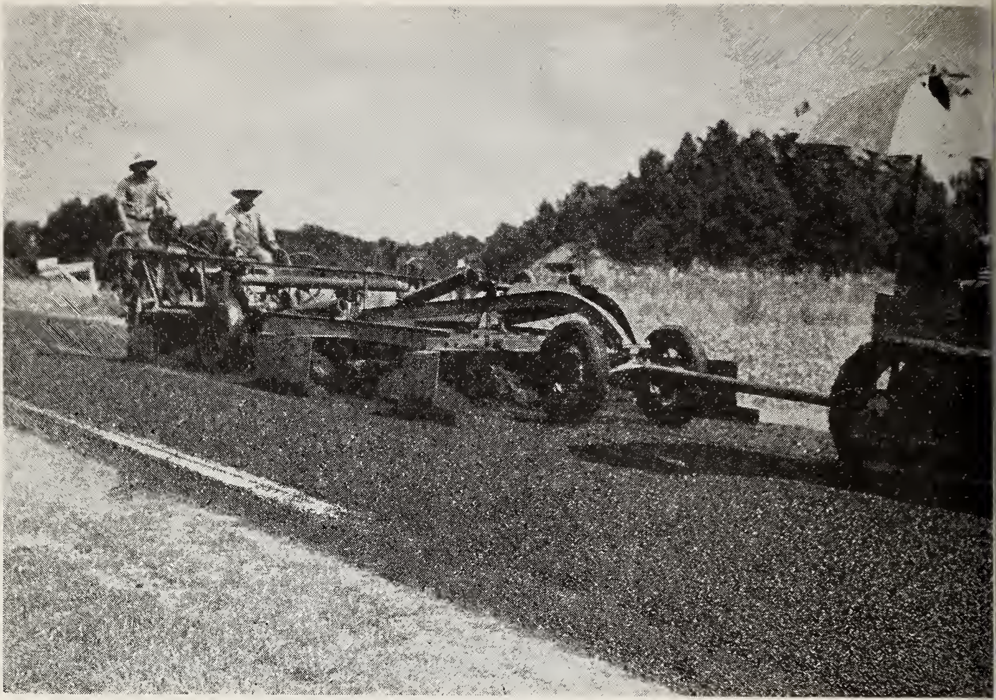
(Iowa 13)
The hookup from the tank car to the pump.



(No. 9)
One of the South Carolina 90 pound treatments. The surface has been shot and the next step is to apply the stone as shown in the picture.

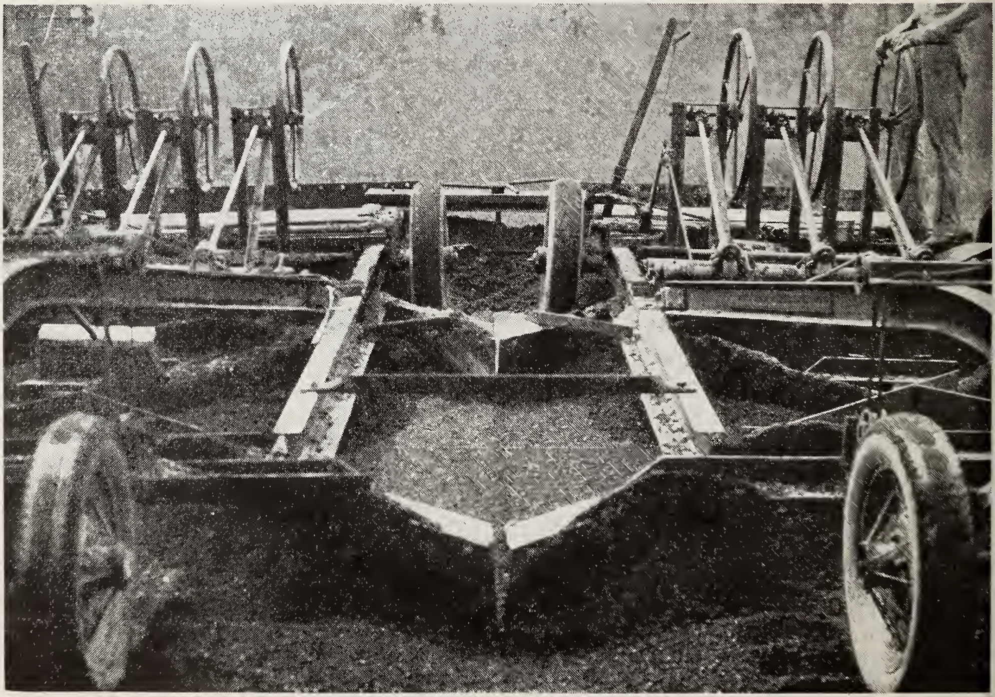


(No. 10)
This shows the next step which is to shoot this rock which has just been placed.



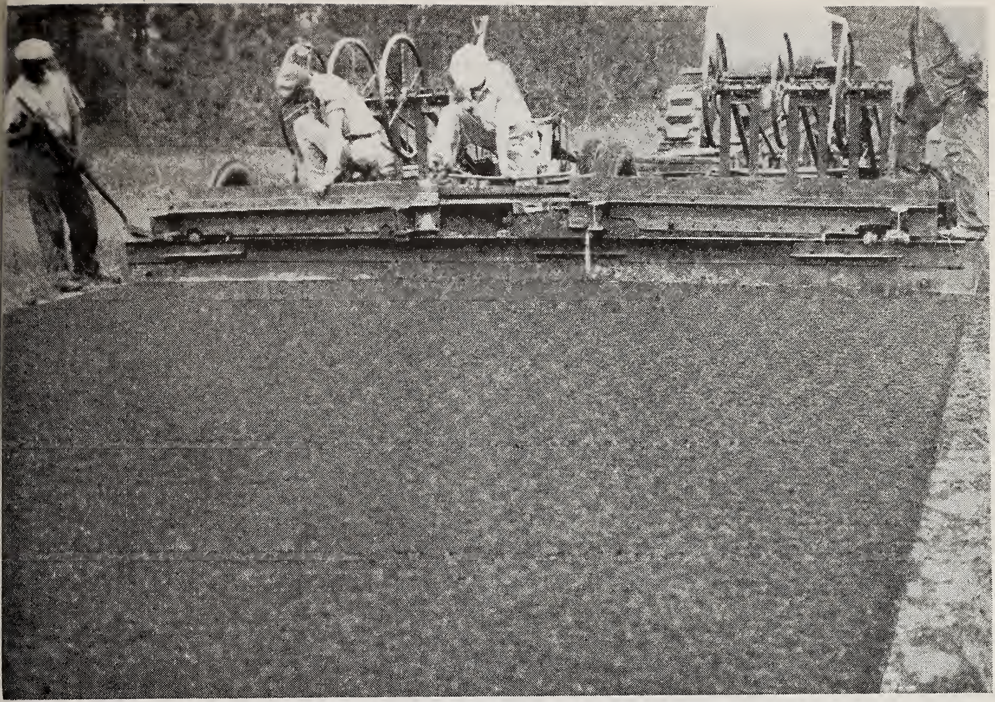
(No. 12)

A multiple blade retread machine mixing this rock and bitumen.



(No. 14)

This shows a combination of two multiple blade mixers which is also used for this same mixing.



(No. 15)

Shows a rear view of the machine shown in picture No. 14. This view shows the back arrangement and the type of surface that the finishing blade leaves.

This concludes my part of the summary and I want to thank all of you for your kind attention.

You men have now had three days of the convention, and I am wondering if there is any argument left. I trust that there is still enough to carry us through this afternoon. To start the discussion, we might take up Mr. MacKenzie's questions as he gave them to us. I am going to ask all of you to be sure to talk good and loud, because you know when anyone has a complaint to make on maintenance work he never talks to us in a whisper, unless it is a whisper that you can hear for about five blocks. The first question Mr. MacKenzie raised was: Does shoulder stabilization and a 4:1 slope eliminate shoulder maintenance? I believe that is substantially the question.

H. R. MacKenzie: Saskatchewan
Largely eliminate.

Mr. Perry: Will some of you who have put down shoulder stabilization take this question? I hope the men who can answer this question are not all out in California or West Virginia.

This year in Iowa, we placed about 34 miles of shoulder treatment. This treatment did not cover the entire width of the shoulder but varied in width from 2 to 4 feet in the various locations, and the depth varied from 6 to 10 inches. We used what might be called a water-bound macadam type of construction. Fair compaction was secured; however, it did not pass 100% Proctor. A surface treatment was placed on top of this base. It is a little too new to really draw any conclusions yet, but from present appearance it appears to be reasonably permanent and is the heaviest type that we have used to date. In the past, we have used a number of penetration treatments along the edge. Some of these were successful, and others have had to be replaced.

Ordinarily, two or three applications of bitumen will seal the shoulders in pretty good shape. I would not say that this prime is a permanent cure, but is does help where traffic does not cut up the shoulder. This light treatment will also eliminate a lot of water saturation or seepage underneath the mat.

What type of treatment has been used out here in these western states? Have any of the conference committee used stabilization on the shoulders?

Levi Muir: Utah

I think Mr. MacKenzie's suggestion of 4:1 slope is really a good one. I know in our state there is very much sloughing off at the shoulder. I notice that many edge failures occurring in Utah are due to too steep a slope on the shoulders, and some of the failures shown here on the screen were of that type.

I was coming into Utah from Nevada on Route U. S. 40, over the mud flat between Wendover and Knolls, and I noticed on the south side of the road a series of patches, and none on the north side. The Western Pacific Railroad parallels the highway on the south. Much of the flat on either side of the road is under water all the year round. The ditches are full of water. It is subject to water very much more than the average road. The water on the south, between the highway and the railroad, is comparatively still, while that on the north comes in with the wind. In constructing the road, the north side was beached with a 4:1 slope, and gravel was placed on the slope to prevent wearing away on the grade. No failures occurred on the north side where they had the flat slope and beached section, whereas on the south side these failures occurred. Undoubtedly, the weight was too much for the soil on 1½:1 slope to support the load, and depressions were created in the surface, which finally had to be patched.

I think the 4:1 slope, or any flat slope, is very much of an advantage and will, in a way, protect the grade a whole lot.

V. G. Gould: Iowa

I would just suggest, Mr. Chairman, that if possible, that question be brought down to a little clearer understanding of what is the source of trouble. Does Mr. MacKenzie refer to edge cuts occurring at the edge of the surface itself, or is he referring to destruction from water, rain water, and so forth, that occurs on shoulders of the road? It seems to me those two might have a bearing as to the probable solution.

H. R. MacKenzie: Saskatchewan
Mr. Perry, I was really referring more to the failure of the outside 15 inches at edge of pavement, due to weakness in the subgrade largely, I think.

Mr. Perry: Does anyone else have anything to offer?

Mr. MacKenzie also had another question: What would you recommend as a relief or a cure for the longitudinal cracks that exist along the edge of the pavement? Mr. Meeker, I believe you have a soil in Dakota which cracks much the same as MacKenzie has outlined. Would you care to tell us what steps you take to correct this difficulty?

E. W. Meeker: South Dakota
This question Mr. MacKenzie brought out in connection with the shrinkage factor of the subgrade materials is very similar to conditions that occur in South Dakota in what we term our gumbo section. The only solution we have found to that is changing the texture of the soil. The high shrinkage factor there will permit this soil to absorb moisture up to 100% of its volume, when the moisture is there. When it shrinks, of course these cracks develop, and will pull the mat right apart and pull these longitudinal cracks to the edge of the mat from 1 to 2 and sometimes 6 feet in from the shoulder.

If you have built your road over this type of soil and have not corrected that soil, the only method of repair is to change the texture of that soil in your base course and subgrade repair. It is a problem in construction, and I think in that type of soil the construction department must take care of it. If you are building a type of pavement higher than even low cost bituminous, we have had experience with concrete on that same type of soil and find concrete won't stay on it. We don't have the longitudinal cracks, but we have the heaving and the breaks in the concrete from 6 to 8 feet from our construction joints. So it is just a matter of making that correction of your soil ahead of your pavement work.

Mr. Perry: Mr. Meeker, your recommendation would be to have the construction department pay more attention to this type of failure and to provide a remedy at that time. I think you are absolutely right in making this statement. I feel sure that the Maintenance Section is not necessarily passing the buck to the Construction Section, but certainly conditions of this nature should be taken care of as a construction proposition, rather than as a maintenance proposition.

This discussion seems to bear out the statement which I made in the summary, that shoulder maintenance and construction has not kept pace with either mat construction or stabilized base construction.

We have in Iowa a gumbo soil which cracks rather badly. We fill these cracks both on the bituminous surface and on the shoulder with liquid bitumen. If the crack is as large as previously mentioned, we would fill the open area with sand and then seal with the bitumen.

Does anyone else have a suggested cure to offer in handling soils that crack in the manner outlined?

R. M. Schwegler:
Oregon
Do I understand that to be a trench section?

E. W. Meeker:
South Dakota
Yes.

Mr. Perry:
Mr. MacKenzie, is your mat placed in a trench?

H. R. MacKenzie:
Saskatchewan
It lays on top.

R. M. Schwegler:
Oregon
Your shoulders are earth, not surfacing material?

H. R. MacKenzie:
Saskatchewan
Just untreated gravel on the shoulder.

R. M. Schwegler:
Oregon
I would suggest that you treat or seal your roadway surface full width, shoulder to shoulder, and also extend the treatment down on your shoulder slopes. The purpose of such a treatment would be to control and attempt to maintain a uniform moisture content in the subgrade which should prevent volume change in the soil and the consequent shrinkage cracks. The cost of sealing or priming the shoulders and slopes would not be excessive.

Mr. Perry:
Mr. Schwegler, do you suggest placing a prime and seal treatment over the shoulder and the back slope, or just the prime treatment?

R. M. Schwegler:
Oregon
Probably a heavy prime.

L. C. Campbell:
New Mexico
With a problem of that kind, it seems there might be three degrees of treatment for such conditions: No. 1 might be called an irreducible minimum which you could get by with as a temporary expedient, which may be repeated; that is, filling those cracks with more granular material and continuing to do that as often as necessary

until the condition has been improved, or more permanent measures taken. No. 2: The top 2, 4, 6, or 8 inches, according to judgment, may be removed and replaced with more granular material as a blanket protection, but in replacing the top blanket you would of course want material which is sufficiently stable to withstand what traffic might get on it. That might be regarded as a more average treatment. No. 3: We could go still further into the more extreme, and call it reconditioning, by complete replacement of material or replacement to such a large extent that it would amount substantially to the same thing.

So you can alleviate those conditions with a minimum of treatment. You can further improve it by a more extended treatment, or you can go to the further extreme and try to eliminate it altogether or to a maximum extent.

Mr. Perry: Does anyone else have anything to offer? If not, we will take up a few topics which were not included in the summary or in the papers presented by the committee, but were included in a number of questions that have been handed up for discussion this afternoon.

The first question pertains to centerline marking. The consensus of opinion of the Maintenance Committee in discussion yesterday was to the effect that Idaho has a mighty fine centerline. Everybody seemed to notice it. I understand that the other neighboring states also have some pretty good centerlines, and a little discussion on centerline marking is in order. Mr. Beckham of South Carolina, and a number of other men, wanted this topic discussed. While they are not here to hear the discussion, perhaps they can get something out of it from the proceedings when they are recorded.

One of the questions is: What is the most economical material for painting lines or guide lines on asphalt surfaces? We probably can divide this question into two parts. One is material, and the other would be the equipment or method of handling. Later on I expect to call on Mr. Flint who is Director of Highways in Idaho and also Vice Chairman of this convention, to tell us a little about the centerline marking here in Idaho. Incidentally, I understand that the Idaho machine will be working near Shoshone tomorrow if any of you drive home that way.

Who wants to make a recommendation on the best type of centerline paint to be used on asphalt surface? Mr. Templeton, what do you have to say?

R. J. Templeton:
Wyoming

I wouldn't care to discuss that. We have been centerlining for the past several years, but I think some of these specialists here know more about the paint they use. I grant that Idaho has a much better stripe than we have.

Mr. Perry:

Mr. Shumate, what kind of paint do you use in Colorado?

C. E. Shumate:
Colorado

I don't know. We might ask the head of the laboratory what kind of paint we are using.

Before we start discussing the way it is handled, or anything else, I would like to say that several years ago, through the American Association of State Highways Officials, we standardized on road signs. In traveling over about six states in the last month, it looks as though we are going to have to standardize on center striping before very long.

In Colorado, this year we started using a solid white center stripe with orange lines at the hazardous points. I notice Wyoming is using a dashed line with the solid line at the hazardous points, using orange altogether. Over in Idaho, I notice they are using a solid orange stripe. In New Mexico, I believe they are now—they were a short while ago—using a dashed white line.

I think that is something this Conference should recommend to the American Association of State Highway Officials at their meeting in Virginia next month, to be taken up by them, acting as the unit they do, and standardize on that phase of traffic protection.

Whether you are right is questionable, or whether anybody is right. Maybe we are all using the wrong system.

As for the type of paint we are using in Colorado, it is called a spirit paint. I don't know what that means, but that is what they call it. It does very well. The paint we are using now we like very much. It is giving excellent service.

As for the method of applying it, I want to hear from Idaho first, and then I will tell you about the thing we have done.

Mr. Perry:

What does your paint cost you a gallon, Mr. Shumate?

C. E. Shumate:
Colorado

I think the last bid we received was $1.18 a gallon, wasn't it, Ken?

K. C. Vail:

That is about right.

Rex Whitton:
Missouri

I might make a short statement relative to the uniform marking of "No Passing Zones," which the gentleman just mentioned. A sub-committee was appointed (of which I am a member) at the Dallas meeting of the American

Association of State Highway Officials by the Chairman of the Committee on Maintenance and Equipment, and a sub-committee was also appointed by the Chairman of the Committee on Traffic Control and Safety; both of these sub-committees to work together as a joint committee for the purpose of preparing a uniform method of marking "No Passing Zones." This joint committee has been working for the past year and hopes to have a recommendation ready for presentation this fall at the annual meeting of the American Association of State Highway Officials in Richmond.

Mr. Perry: We will certainly be interested to see what the committee brings out. Could you give us a little inside information now, Rex?

Rex Whitton:
Missouri

Mr. E. W. James, of the Public Roads Administration, is chairman of the joint sub-committee, and he has just recently sent a letter to all members of the joint sub-committee, in which he indicated that the most desirable method of marking "No Passing Zones" seemed to be by the placing of an auxiliary line on the pavement to the right of the normal line; or, in other words, in the lane of traffic that it is to govern. He also indicated that it might be desirable to have the additional stripe of greater width than the centerline stripe. This is about as far as we have proceeded with this matter. I have replied to his letter, and my own personal recommendation in the matter is that besides the additional width of the "No Passing" stripe it should be of a different color.

C. E. Shumate:
Colorado

I might add that in Colorado we are using this year the idea of the double line at the hazard. The centerline is white, the outside line is orange, and it is in your lane at the hazard. The wording of that sign is beginning to bother us considerably because it is quite elaborate, but that sign says: "Do not cross the yellow line when it is in your lane." The public has taken very, very well to it, and I believe that is the method you say has been recommended, is it not?

Rex Whitton:
Missouri

That is the one that is being discussed. I might add further that the explanatory sign, in my opinion, will become unnecessary as soon as we establish a standard method of marking.

C. E. Shumate:
Colorado

Unquestionably it will.

Rex Whitton:
Missouri

Of the 48 states, 41 states have methods of marking no-passing zones which are of two different general types. There is the shoulder type of marking, and the pavement type of marking. Under the pavement type of marking there are also three subdivisions, in that some states have no passing line in your lane, some have it in the opposite lane, some have double lines to indicate no passing, and then some have apparently a dashed centerline on straight-aways and a solid line over the hill.

C. E. Shumate:
Colorado

That method is what prompted the system we are using now. Our first center striping was just in the hazardous locations, and warning signs were placed, "Do not cross the yellow line." A motorist, after following a heavy truck on a half-mile hill, after reaching the crest of the hill and looking down the hill, sees ahead still another quarter-mile of yellow line but was very much tempted to cross the yellow line, which he had a right to do. Our state police force was so efficient that John Public put up an awful fuss and, in fact, it got so bad we had to develop something else. The system we have now seems to be working very well and I only hope, as Mr. Whitton says, it is standardized so it will be the same all over the United States, the same as our traffic markings are.

Mr. Perry:

Does anyone want to say a word on the proper kind of paint to use, that is, whether you should use an asphalt paint of various colors, a lead paint, or lacquer paint? A number of the members wanted a good hunch as to paint that will wear well, one that can be easily applied, and one that will not track. This is quite an order, yet it is exactly what is needed in a first-class traffic paint. Has anyone experimented with these various kinds of paint?

K. C. Vail:
Colorado

I think the formula for the paint that has been used in most of the Western country was made in California. It is a fast drying paint. We use chrome yellow for the yellow color. There are other paints that would last much longer on the pavement, but there doesn't seem to be any object in having something that will last too long, because you tear it up and it gets dirty. What you want is something that will chalk off a little bit and keep bright.

I think most of the specifications ask for about twenty minutes' drying time; otherwise, it would be scarred up with automobile tracks. Of course, with such a fast drying time you can't have as long life on the paint. A vehicle has to be set so fast that it doesn't have the staying quality.

H. G. Nevitt:
Missouri

Mr. Chairman, I would like to ask if the idea developed some years back and perhaps tried out, of a raised stripe in the center, using an asphalt with either a special aggregate or an aggregate which stood above the level of the pavement so that you can see it or even feel it, was found sound; that is, whether anybody has really developed the idea enough to have any findings on it, any record of performance, like or dislike.

Mr. Perry: I believe that the development which Mr. Nevitt mentions was tried out in some of the southern states, and I wonder if there are any members present from that section of the country who could answer Mr. Nevitt's question. Apparently, we do not have anyone who can give you this information, Mr. Nevitt.

If Mr. Flint from Idaho will now tell us about the centerline, we will be glad to hear how they do it.

H. R. Flint: Idaho

I will start the discussion. There are a couple of men here on whom I may call, if the committee so desires, to explain a little more in detail.

Last year Idaho started the double line through the hazardous places for the first time, and as some of you may have noted, there still exists a double yellow line. Our experience with John Public is similar to Colorado's. This year we revised the method, carrying our solid line clear through and placing a second stripe of white which, by the way, appears to be narrower than the yellow one, but is supposed to be the same width. Possibly it would be better if it were wider. The white line is carried to the point where the driver can see ahead and thus avoid breaking the law. The public has taken this up very quickly. The law enforcement officers have cooperated in it. We have given it publicity, and it has secured the good opinion of the public at large.

The cost to us for last year, I believe, was $26.41 per mile. That included rental for the state equipment. As to the cost of the paint and the type of paint we are using, I would prefer to refer that subject to Mr. Hallvik, our materials engineer, and if you wish any discussion on the operation I would like to have you call on Mr. Haight, district maintenance engineer of this district.

I would like to announce further, relative to the operation tomorrow, the foreman of the crew immediately in charge will be here—he was here Monday to listen to the traffic engineering discussion—and he will be glad to offer any help that may be wanted on the operation. Tomorrow, they will be between Shoshone and Gooding. That will be to the west of Shoshone. If it is raining, the equipment will be stationed at the district office at Shoshone and may be observed there.

Mr. Perry: Mr. Hallvik, do you want to say a word or two in regard to the paint?

Clif Hallvik:
Idaho

Our specification for the paint is, I believe, a conglomeration of specifications from most of the western states. I imagine California had most to do with it. We changed this year in an effort to get a better product. I believe our cost is $1.20 or $1.24 a gallon, f.o.b. destination. We have it delivered at five different points in the state. We apply an estimated amount of about nine gallons per mile painting on old surfaces, and up to thirteen or fifteen gallons on new treatments. Seal coat, especially, takes more paint.

Mr. Flint gave you the cost per mile, which is quite comparable to what I imagine it will be this year. Originally, they used to get a transit and try to run a straight line down the road. It didn't work very successfully, and now they use a string, and maybe they have a mile of string stretched out ahead of the paint machine. After they get the line painted once, the painter next season can follow the line painted before.

I don't know of anything else to add, except that last week I talked to the testing engineer at the Nebraska University, and I believe he said they were using a lead paint, and were working on a paint that dried from the top down. That would seem quite plausible, if you can work it out. Anyway, they felt they wouldn't have to have such a fast drying paint if they had something that would dry from the top down, so the tariffic wouldn't mark it up and it would remain alive underneath. If there is somebody here from Nebraska, I would like to hear more about how it holds up.

Mr. Perry: Mr. Haight, can you say a word regarding the painting operation?

H. G. Haight:
Idaho

Running the paint line isn't complex, but it consists of a considerable number of details.

The paint machine is a four-wheel apparatus about eight feet long and about three feet wide, with a seat for the operator, and a long sight rod, about eight feet, extending in front of the front wheels. At the front end of the sight rod there are numerous vertical rods which can be adjusted by the operator to follow the string mark, re-mark, offset lines or curved lines. The operator, by getting used to these, can switch to any of these vertical rods, which are about eight in number. It is obvious, of course, that in following a curve or making a curve, he couldn't follow the string; he has to follow an offset; By switching to the various horizontal bars, he can follow a very definite course.

This four-wheel machine is propelled by a truck, and on the front of the truck we have a long horizontal slot which allows for side variations of about six feet. The truck doesn't necessarily have to follow directly behind the machine; it can follow at an offset. This is particularly necessary on curves.

The truck carries all the paint, the machinery for the air compressors, large paint pots, and the cleaning and washing apparatus, which consists of a thinner which comes with our lacquer paint. We use a lacquer with a heavy gum content.

The line is laid down first by the string method. They go along and mark an offset on their pavement 0.4 from the center. The string is laid directly over the line by the string man and is pegged down every 550 feet by nailing. Intermittently, rocks are laid on top to keep it from blowing or shifting until the machine gets to it. On curves, this is done entirely by sight. They start at the back tengent, lay it around half way, go around to the head tangent and lay back, and bring the curve together in the center. There is usually an excess of string there that is wadded up. The string is wound up by a string man. We have a two-strand string similar to binding twine. We line is out and reel it in each time to save the string from excessive wear and to protect it from traffic.

You can make about nine miles a day on newly laid striping, 16 to 20 miles on re-striping. Double striping doesn't require any more time, except that the operator needs to be more skillful in operating the triggers on the machine.

We have three guns with four disks, two disks for the center stripe, and these with two outside disks for marking the double lines.

We use Crown Spray guns No. 10 We can use up to 80 pounds pressure, although we usually work around 40. If we want considerable speed on remarking, we can raise the pressure and get up to around 20, sometimes 25, miles per day, if we have especially good working conditions. In windy weather we can't work with string, but we can remark if we have anything in the vicinity. In wet weather we have to lay off entirely.

In keeping traffic off the line, we use only signs. We used to use flagmen. We are using signs only now, and it doesn't cost as much as for the flagging. Women seem to be the most obstreperous persons on marking up the lines. They don't seem to think it is any different than any other part of the road.

I am of the opinion that the solid line may have advantages, but I think the broken line would have the advantage of less trouble from marking up and woul cut down the quantity of paint used about one-third, a least. Our center line marking costs us $26 a mile. Sixt per cent of it is for material. I believe by using a broke line of about 25-foot segments, we will lower our cos about one-third.

I think using the solid line on dangerous section should be done in all cases. I know that in snowy, rain or foggy weather, or in night driving, that yellow lin is a welcome visitor wherever you see it.

Mr. Perry: Mr. Haight, this is very interesting. Is the machine propelled ahead of the truck or is it pulled?

H. G. Haight:
Idaho

Propelled ahead.

Mr. Perry: What do you do, or what steps do you take in regar to the safety of the operators? It seems that in man places there is a sort of open season on these men, an they are in about the same position that the early aviator were who operated the first Wright planes. They sit righ out there in front where they have to take it.

H. G. Haight:
Idaho

Two years ago we had flagmen both ahead and behind and they stopped nearly every car or truck that came by At the present time, on striping work we have two me at the rear. One is mostly a service man who picks u the sign, the twine, and so forth, and we also have man at the rear winding the twine. Ahead we have th string man, the rock layer, and the foreman of the jo is usually out ahead, and also the service man make regular trips up and back to deposit the string and material to rock the string down. We try to warn th public by signs and flagmen. We have used only sign this year and have had no serious accidents. We hav tried to have the state police, wherever available, aid us in some of the heavy traffic sections. We have had no fatalities that I know of.

Mr. Perry: Do you paint during the day or during the night?

H. G. Haight:
Idaho

Altogether in the daytime.

Mr. Perry: I might add a word in regard to night painting. In Iowa we have been center-lining our concrete pavements at night. The men start to work about 11 o'clock and finish about 7 in the morning. The reason for painting at night was to get away from the traffic and, of course, eliminate tracking of the paint as much as possible.

We have placed everything imaginable on this center-line marking truck; yet traffic has driven into it from both directions. We may have to go back to daytime marking. The carelessness of night drivers is rather a serious proposition. Not so long ago, I rode one of these paint trucks and I do not blame the men at all for getting off the truck the quickest way possible or making a sharp kink in the line the way some of the cars come at them.

H. G. Haight: daho

In regard to marking, you say you have everything you can imagine on the truck. That is the way our truck looks. It looks like a traveling sign painter's truck: ''Pass to the right of the line,'' ''Keep to the right of the truck,'' ''Go slow,'' ''Watch out for men and equipment,'' and so forth. When you ask some of these people to pass to the right of the truck, they stop and argue with you and say it is against the law to pass a vehicle on the right, and proceed to go around at the left. It brings up quite a few difficulties. We are covered with flags and signs, and the operators are careful in warning everyone. That is how we get by as well as we do. We sometimes start at four or five o'clock in the morning to get ahead of traffic, and work through the cool of the day. We find that is not so good because the paint doesn't dry. A long string of pavement behind you keeps wet. As the weather gets hot, you don't have much trouble with the drying.

Mr. Perry: We even use a swinging signal on this truck. It helps very materially. Mr. Shumate, have you anything to say about your marker?

C. E. Shumate: Colorado

Maybe Idaho has one on their marking machine. Ours is very similar to theirs. We are, however, using the transit line, especially in the eastern part of the state where we have long tangents. We found it greatly speeded up our work.

We have a big free-rolling siren on this paint machine. As Mr. Haight said, it looks like a circus wagon coming down the highway, but that young man who rides out in front has made several detours this summer already; in fact, one day he just walked off and left the truck. I don't believe it has been hit yet, but that was last week; it might have been hit by now.

As far as costs are concerned, I am very much interested in hearing Idaho's experience, because our center striping is costing about \$27 to \$28 a mile, and we felt probably that was in excess of what other states were doing. In fact, we heard rumors that other states were painting for about \$16 or \$17 a mile, but our costs are running \$27 to \$28 a mile.

S. B. Sanders: Montana

We use a paint machine mounted on a flat rack, ton and a half truck. We make about 25 to 30 miles a day with this outfit, using a transit line for a guide. There is a sighting attachment mounted on the front end of the truck. Of course, the operation depends upon the skill of the driver.

The cost per mile ranged from $10 to $15. We use the solid line on the hazards and broken line elsewhere.

We have considered many different kinds of paint. I am not prepared to discuss the physical properties of the paint. We use a white paint that dries in about ten minutes. We use a flagman in the rear, and this season and last we have had the cooperation of the state police, and have had very little trouble from the line being tracked.

Mr. Perry: We are spending quite a little time on centerline marking, but I believe it justifies rather detailed discussion this afternoon, because the general subject seems to be requiring considerable attention.

R. J. Templeton:
Wyoming

The cost of centerline painting in Wyoming is approximately $10 a mile. I think that is one of the questions you wanted answered. We use a broken line, as the gentleman from Idaho said, to cut down the cost. That is the only reason we use the broken line—to keep down the cost. We double stripe around curves and over hills with short distances. We use the orange or yellow lacquer to give a color contrast. We find this a very good idea for the winter months. During the winter months we have a lot of what we call ground blizzards, and when you are driving, about all you can see is the centerline. Drivers often have to follow the centerline. I don't mean they straddle it, but they follow alongside, and that is the reason we have used the orange or yellow stripe.

J. G. Schaub:
Michigan

What material do you use for striping concrete?

Mr. Perry: In Iowa we use a black tar paint and also an asphalt paint for striping the concrete pavement. The tar paint costs us about 16 or 17 cents a gallon, and the asphalt paint a little more The fault with the asphalt paint is that it does not dry as fast as the tar paint. I am not trying to sell either one, you understand, but simply telling you our experience. I want all the rest of you to feel free to say just exactly what you think in these discussions. We still feel there are some possibilities with the asphalt paint, but so far it has not worked out so satisfactorily.

H. G. Haight:
Idaho

Has anyone had any occasion to use casein paint? Another question is, what do the field maintenance men think about center striping when it comes to kicking their shoulders out when the traffic is at the edge of the road?

Mr. Perry: Has anyone tried the casein paint? Evidently no one here.

Does anyone have anything to say about the advisability of using centerline marking on the bituminous surface? Mr. Haight brings out the fact that splitting traffic creates an additional destruction to the outside edges of the mat.

R. J. Templeton: Wyoming

Mr. MacKenzie mentioned this afternoon that the center stripe had better be left off. We have a road in the northern part of Wyoming commonly known as the Colony Road. I believe Mr. Sanders is familiar with the shoulder condition on that road. He has a similar stretch of road conecting up with ours. We feel that if we center stripe that road we would have the traffic riding out on the shoulders, which would cause a lot more trouble than we are having now. We have intentionally left the stripe off.

C. P. Humphrey: Idaho

I come from the northern part of Idaho, the bone part of the pork chop. We have a good many miles of very narrow grade on our north-and-south highway where there is just about room enough for an oil mat, and very little shoulders. We have purposely left the center stripe out on some of those sections feeling it would make the driver drive too close to the shoulder, and we would also have heavy maintenance. The demand from the public has been so insistent that we center stripe those stretches that we have had to do it.

Although I have no costs available to definitely state this, I believe I can say it does increase, on that type of narrow grade, our shoulder maintenance and our maintenance on oil.

Mr. Perry: There probably isn't any question but that a centerline does tend to split the traffic. After all, maybe that is what it is for; there could be that possibility. Perhaps the construction departments should give us a wider road so that we can mark them. I know Mr. Moyer wants a wide road and wants it centerlined. On these older narrow roads it is quite a question as to whether they should be marked or should not be marked. We have some in Iowa that we do not mark for the simple reason that maintenance costs are increased when the centerline is applied.

If there is no more discussion on centerline painting, we will pass on to the question of shoulder maintenance. Any discussion that we may bring up here this afternoon on this topic may be a slight repetition of topics covered in the previous three days. The question has been asked whether or not vegetation on the shoulders is prohibited, and also if vegetation is allowed to grow, does it damage the shoulders? Mr. Sanders asked this question, so I am going to ask him to discuss it.

S. B. Sanders: Montana

We haven't done very much in Montana on shoulder work, that is, in regard to vegetation control. There is a question among ourselves as to whether it is really worth extensive expenditure. We have in construction in my district right now—I expect the project will be completed by the time I arrive home—eleven and one-half miles. We have a 24-foot mat and we are priming the shoulder with SC-2, followed with an application of 0.3 of 95 plus, and the regular mat material aggregate. In addition to the shoulder itself, we are applying the 95 plus down over the slope about 2 feet. It is questionable in our minds whether the 95 will obstruct the growth of vegetation, but in our state and in many of the Western states we have a heavy growth of tumbleweed or what I think is generally referred to as Russian thistle. They are most objectionable to us in snow removal. By some means or other we have to see that they are removed before the winter season sets in, because they make a very neat snow fence along the shoulder, and if we allow these weeds to remain we consequently get drifting.

In that same connection, we think perhaps these weeds collect and hold moisture. The vulnerable point of a road, as far as moisture is concerned, of course, is the shoulder, and these weeds certainly do grow and thrive along there. Since they undoubtedly absorb a great deal of this moisture, we think perhaps they do collect and hold enough of it to influence an unstable condition of the shoulder, and perhaps also the oil mat itself.

This is only an experimental job. It is the first one in the state, and we believe we are on our way toward controlling vegetation along the shoulder line with bituminous treatment. However, we don't know what the success of the thing will be. As far as I know, in our state this is the only type of bituminous treatment that has been applied with this in mind.

C. E. Shumate: Colorado

We apparently are working at cross purposes entirely, because in Colorado in the last couple of years we have gone into extensive planting of sweet clover on our shoulders and slopes. I am beginning to doubt now, after hearing Mr. Sanders. We were laboring under the impression that probably that growth along there would pull moisture out of the subgrade instead of bringing it in. I don't know; maybe we had better plow it all up next spring.

Wherever it is possible, we are keeping our shoulders and slopes mowed. We have had remarkable success with sweet clover, especially in the eastern part of the state where the erosion from wind and water is quite severe. The success we have had has been remarkable. We have entered into several cooperative projects with the soil conservation service, and in these projects we agree we will do nothing to disturb that vegetation other than to keep it mowed. We have found, on one section of Portland Cement concrete pavement where we planted sweet clover and have kept it mowed down almost flush with the pavement, that that shoulder has given us absolutely no trouble, even in the wettest weather. Traffic is able to pull out on to it without any serious trouble. In those sections where we have been unable to get anything to grow, we have the same trouble with the Russian thistle, and its snow fence properties are unquestioned. I am very anxious to see how Mr. Sanders' experiment works out there, because there are some sections where it is impossible to get any type of vegetation to grow, and if we can prevent that erosion it is going to materially reduce our maintenance costs, especially in the Western states.

J. G. Schaub:
Michigan

I might say that in Michigan we have about thirty inches of rainfall a year, and our climatic conditions are much different, but we deliberately seed and prepare our shoulders for a growth of grass. We have very little trouble. The traffic can get off and on, and we have no difficulty. On our more heavily traveled sections, we use gravel shoulders and place some bituminous edging around the curves, but we deliberately encourage the growth of grass on shoulders all over the state, except in the heavier traffic sections.

Mr. Perry:

Do I understand, Mr. Schaub, that you encourage the growth of vegetation both on the bituminous sections as well as the concrete sections?

J. G. Schaub:
Michigan

Yes, on all types.

S. B. Sanders:
Montana

Mr. Perry, we had a dual purpose, of course. I perhaps didn't bring it out clearly that our primary reason for treating these shoulders with bituminous material was to waterproof and try to obtain an impervious mat and over the shoulders, because in our state we have so much trouble with gumbo soil which I know a lot of you perhaps aren't familiar with in the Eastern and Middlewestern states. These boys in the West—Mr. Templeton and Mr. Meeker—know what gumbo it. We are trying to maintain a blanket over this gumbo so that the water will shed over it instead of going into it and being absorbed by it. Of course, this vegetational growth control is sort of a contingent idea but we hope to accomplish some definite results.

Mr. Perry: I am also concerned with this question of vegetation on the shoulders. This year has been a very good growing year in Iowa, and the blue grass and clover has grown up on the shoulders tight to the stabilized bases. In other years, we have primed these shoulders to shed water. The nice growth of grass we have been getting this year makes me wonder whether it is going to be detrimental or not. I rather feel that we will probably go along with Mr. Schaub and try it out. I like a green shoulder of blue grass and clover, and also the looks of it right up tight to the base or the edge of the mat.

H. R. Flint:
Idaho
I would like to ask Mr. Schaub a question. In planting a lawn, I would very carefully have my ground below the sidewalk so that in sprinkling I wouldn't hamper the foot traffic by the sidewalk being covered with water. In a few years the accumulated sod would have raised the elevation of the lawn above the sidewalk.

How does that perform on the shoulders of your road, and if it does, how do you remove it?

J. G. Schaub:
Michigan
I would like to have you ask Mr. Stokstad to answer that. We leave the shoulders slightly low to allow for swelling in the growth of the sod. Mr. Stokstad has paid particular attention to this type of planting, and I think he has something to tell you.

O. L. Stokstad:
Michigan
New shoulders do shrink considerably the first year or so due partly to erosion but mainly through the compacting action of traffic the consolidating effect of drying. As the shoulder sods over, however, the vegetation catches and holds migrating soil materials. It is possible that sand used in treating icy pavements contributes to the material held by the grass. The Maintenance Division finds that it is occasionally necessary therefore to lower the shoulders to maintain proper surface drainage for the metal.

H. G. Haight:
Idaho
I notice in Idaho that by encouraging the growth of both weeds and grass on our roads, we have also brought in all the loose stock in the country. We are going to fence sections of the road to see if that will stop the loss of a considerable number of stock by our stockmen. I think the growth of all vegetation should be kept from the shoulder at least, because that brings the stock right to the traveled way of the road. When they get up there they begin to rub down the sign posts, and so forth. It is altogether a disagreeable proposition.

H. G. Nevitt:
Missouri
I have a safety question. If you grow sod there, will the traffic go out there? They may go out there to park,

but will they go to change tires? If you give it bituminious treatment, the man feels he can't jack a car on that. Every time I have seen a car changing tires, if it is the outside tire they seem reluctant to get very far off on the sod to work on it. I wonder if this California practice of giving a bituminous treatment on the side isn't a safety measure, as Professor Moyer brought out, because they will go out there to change tires, and so on, if they can.

Mr. Perry: I think this is particularly true in the winter season. Does anyone else have anything to offer? If not, we will pass on to a question that has been brought up in regard to spring break-ups or frost boils. The question is: What seems good practice in maintenance of bituminous mat failure over frost boils, and when should the repairs be made following the failure? Mr. Meeker, do you want to enlarge upon this a little?

E. W. Meeker: South Dakota

The information is just this, that there seems to be a difference of opinion as to the time of repair following break-up on frost boils. What I mean by that is that I have seen some maintenance crews starting to patch a failure in a bituminous mat as soon as it appears. That became disagreeable to the traveling public. I have seen them maintaining this same failure for a month or six weeks, or two months. That is the proposition I want to bring out in the question.

Mr. Perry: Has anybody a recommendation to make on this proposition? Any of you who have had frost boil break-ups can clear up this question for Mr. Meeker. No doubt, there is a question as to the proper time to take care of repairs caused by these frost boils.

C. E. Shumate: Colorado

I would like to say this much in connection with them: They are about the most disagreeable things you can have from the standpoint of maintenance, but the fact remains that regardless of what kind of job you do or how much it costs, it has to be taken care of immediately because it is distinctly a traffic hazard. They have an unhappy faculty of occurring right square in the middle of a perfectly smooth stretch of road and, from the standpoint of traffic safety, we spare no expense to keep them as smooth as possible. Sometimes, as Mr. Meeker said, we spend several days over a period of several months fighting that one boil, but it is a traffic safety measure which we feel is our direct responsibility regardless of cost.

Mr. Perry: May I ask a question in connection with frost boil repairs? Do you try to permanently cure the frost boil at that time, or make temporary repairs and then later follow up with a cure?

C. E. Shumate:
Colorado

We don't try to permanently repair it until such time as all the frost has gone out of the ground and it is dried up. Then we attempt to go in and repair it so it won't occur again.

I have in mind one in particular that occurred right in the bottom of an underpass, and I imagine we spent $200 on that one spot, and it wasn't over 15 or 20 square yards in area. We worked on it for a period of six weeks or two months, doing everything we could to keep it smooth. After the frost had all gone out of the ground and it had dried out, we went in and excavated the subgrade material and fixed it. I don't know whether it is fixed permanently or not ,and won't until next spring, but we hope it is.

Mr. Perry

That seems to be a common place for this type of bulge to occur; that is, on the shady side of overhead crossings. We have a number of these in Iowa and they are a serious proposition.

H. R. Flint:
Idaho

I may be bobbing up here too often, but maintenance is one of my babies and one I lived with for seventeen years, and I love it.

I would like to know what is done to eliminate the frost boil. Is there some way of draining through the frost layer underneath to hurriedly eliminate the nuisance?

C. E. Shumate:
Colorado

I might suggest one thing that I wouldn't advise anyone to try, and that is to thaw them out with steam. We tried it once.

R. J. Templeton:
Wyoming

Do you mean a permanent cure or just a temporary one?

H. R. Flint:
Idaho

A temporary cure.

R. J. Templeton:
Wyoming

I think you have just about the same answer as all the rest of us have.

H. R. Flint:
Idaho

I have heard men in the field offer various solutions. One of them happened to get his truck stuck in one of them. In getting out, he got a hole punched through the frost, and he noticed that the boil dried up much quicker than in any other place.

I had heard the steam proposition offered, but we never tried it. Frankly, I was afraid to. I have also heard the theory advanced of punching a hole down into the frost, then doing something that might be detrimental, adding a half stick of dynamite and shaking that stuff loose. That is something I haven't tried, either.

Mr. Perry:

Has anybody here ever played with dynamite in these places?

I might tell you of two or three experiments we performed on some of these cases, and I think I can tell you this without any fear of contradiction on the part of the calcium chloride people. A few years ago we bored some eight inch holes about 4 feet deep and 5 feet apart in one of these frost boils and then filled these holes with a fifty-fifty mixture of calcium chloride and pea gravel. The results were apparently very good. The next spring water seeped out of these holes, and we had no break-ups.

On another hole that was even worse than the one just mentioned, the foreman attempted to use the same treatment, but he found that he did not have any chloride so he put in straight gravel and it worked just the same.

On the calcium chloride treated sections we have considerable settlement due to the fact that, after calcium melts, depressions occur in the surface and it is necessary to go back several times to fill them up. Perhaps we should have used a brine instead of the flake crystals.

You can smooth up a frost boil temporarily, but it will come back in many cases and your trouble is just repeated unless you correct it. Last year we went into the proposition of frost boils rather generally and made quite an extensive survey of these troublesome spots. By borings we found that some of them could be cured by drainage. Oftentimes a sand pocket existed just below the surface of the road which could be drained out very cheaply. In other cases the soil was not the kind that would drain readily and additional base had to be added or soil substitution made. But the big advantage of the survey, to me, was that it gave you an idea of how to attack the proposition instead of going at it blindly. I want to say to you that you can well afford to spend considerable time trying to find out what causes these heaves and break-ups, and also that when you do find out, it will save you money in the long run.

I recall that years ago we put in a lot of tile, some of which, I am now satisfied, never have carried a drop of water and never will carry any. You can readily see that in such a soil a frost boil cannot be cured with tiling.

Does anyone else have anything to say in rega to frost boils?

The next question deals with winter maintenance. What methods have been used in cold weather for winter patching? Mr. Muir, I believe you brought up this question, and are you satisfied with the statements made in the summary, or do you want to have it discussed a little more?

Levi Muir: Utah

In our state Mr. Miles—by the way, he is here—of District No. 1 in Utah, on U. S. No. 30, has had quite a few breaks, and I think he has made use of emulsions pretty well for that purpose. He might tell you about it.

F. D. Miles: Utah

About all we used was Bitumuls instead of the cutbacks. Due to the fact that the weather was so cold, we didn't dare heat the cutbacks with open fires.

Levi Muir: Utah

There is another point that has been worked out, and that is the drying out of the patch as a whole with a blow torch, which makes it possible to put in some pretty successful penetration patches in late winter and early spring, even with cutbacks. I think that has worked out pretty well in some cases.

Mr. Perry:

I think Mr. Blomgren of Michigan also brings this out pretty clearly in his paper.

H. R. Flint: Idaho

This is a bituminous convention. However, I think I might mention another product without being thrown out. We have in this western country a product known as Raylig. It is lignin, a by-product of the pulp industry. I have used that somewhat in this respect, and with fair success, especially in the spring of the year when it is too wet to put in a bituminous mix. We know that gravel won't stay, so we take a can of this lignin along with a load of gravel—there is always water available. This lignin is in concentrated form, so you add 50 per cent water with it, mix your gravel in this lignin, slap it in a hole, and it will stay longer than the gravel will, and will sometimes stay long enough so that later on you can even come along with a little flush coat of oil and sand. I believe some of those patches we put in are probably still in the ground after two years' service. It is one way of using something better than just gravel.

Mr. Perry:

Mr. Flint, this is very interesting. Has anyone else here used this same material or a material of this same nature?

J. G. Schaub: Michigan

We have experimented a good deal with pre-mix patching material and have developed a mixture we like. We used SC-4 oil and make up what you might call a bituminous concrete mixture. We use about 50 per cent of three-eighth inch stone aggregate and sand, and some filler. The mixture requires about 6 per cent of SC-4. We store some of this mixture in our maintenance garages. If the weather is not too cold, it is workable, due to being stored in a warm garage, for emergency patching in the winter. If the weather is cold, we have hot water or steam jacketed heating pans which our crews use to take care of winter patching. Of course, you can't patch successfully when it is wet. A torch to dry the area and bituminous prime coat material are generally required.

Mr. Perry: How about conditions up in Canada, Mr. MacKenzie? From the weather reports, I understand that it is rather cold up there in the winter. Do you patch in the winter time.

H. R. MacKenzie:
Saskatchewan

No, Mr. Chairman. We have had to postpone maintenance operations in the severe weather. We haven't much difficulty with holes developing in the winter. The surface is covered with a certain amount of snow and the maintenance work can satisfactorily be postponed until the spring break-up. That is when our difficulties come in.

Mr. Perry: I believe Mr. MacKenzie is fortunate. Has anyone else anything to say in regard to winter maintenance?

I might say just a word here in regard to the desirable width of the mat from the maintenance standpoint. You will recall that, in the maintenance discussion and also in some of the other discussions, mention was made of the proper width of the mat from a maintenance standpoint. These construction men might be interested to know just what our recommendations are. The question is: What is the desirable width to have? In other words, tell the construction section just how wide you maintenance men want these roads. Mr. Muir, do you want to make the first recommendation?

Levi Muir:
Utah

At the present time Utah is building mostly two-lane roads, each lane being 11 or 12 feet in width with shoulders varying from 4 to 8 feet in width.

In the discussion of vegetation on the shoulders and edges, I think there is one thing that might have been neglected, and that is conservation of the shoulder material. As soon as our landscape engineers commenced working, the first thing they demanded was to let the vegetation grow on the shoulder. I think they tried to plant some and didn't have any success. There are a lot of places where Russian thistle and other weeds grow on the shoulders, and that works out pretty well. If the lanes are wide enough, the traffic doesn't get out on the shoulders and cause any trouble. On the other hand, if the road is too narrow, the vegetation will be destroyed by traffic. I think in practically all cases now we are going to a wider width with the flat shoulder, and I think in most cases the grades we are building are wide enough for three 11 or 12 foot lanes—that is, the newer roads—although where we put pavement on, or oil, we have only the maximum now of 22 feet.

Mr. Perry: Mr. Templeton, would you care to enlarge upon this question?

R. J. Templeton:
Wyoming

Mr. Chairman, when we get to talking about these maintenance features, we kind of step on the construction boys' toes sometimes. Mr. Muir brought out that they are extending the width of their mats, I think he said, to 11 feet for each lane. Our later roads are designed for a 24 foot mat; that is, 12 foot traffic lanes. Our shoulders, on recent construction, are about 6 feet. In other words, we are getting about a 36 foot shoulder-to-shoulder road, with flat slopes. Along our light fills the slopes are carried out on a 5:1, and I think next year they will be on a 6:1. The heavier or deeper fills are, of course carried out to a 1½ or 2:1, and around the 6 feet depth they run about 4:1.

Recently, to eliminate some of the trouble we were having on roads with mat width not sufficient to carry the traffic without running over into the gravel shoulders, we have extended our mats clear out to the edge of the shoulder. That is only on older roads which were constructed to a 24 foot width standard from shoulder to shoulder. That works out very successfully where you have good soils or good bases under your mats, but where you have the clay soils around 6's and 7's you have considerable trouble, because the traffic seems to travel the entire width. If you oil the shoulder, they are going to travel it. We have considerable trouble where our soils are poor; where they are good, everything works out fine.

In some places where we put on a stone chip treatment, we seal over the chips. If we leave the seal off the top of the chips, we seal the shoulders so as to give a color contrast, to keep the traffic off the shoulders as much as possible.

In extending the mat over to the shoulders, we feel we are keeping the water from getting under the mat, which does destroy the stability.

Mr. Perry: It might be a good idea to stop right here and give you men down in front a chance to ask a few questions in order to clear up any troubles that you may have.

Does anyone in the audience have any questions that he would like to have discussed regardless of whether it has been discussed before or covered by any of the summaries? This is like throwing out a rubber hook, which I understand they do over at the small lake.

Mr. Meeker brings up this question: Is a thickened edge on a bituminous mat of any maintenance value? I know that some of you have been using a thickened edge section and I would be glad to have you tell us whether you are continuing this practice.

Clif Hallvik:
Idaho

Several years ago when we began oiling, of course most of the grades were built with trenched sections. The surfacing was put inside the shoulders for a depth of 6 or 8 inches, whatever it happened to be, on the grade. When they went out to oil the roads, the feathered edge was always breaking down, and naturally the thickened edge was provided. It has proved to be very successful.

In the past two years we have discarded the trench section and have gone to surfacing bases from shoulder to shoulder. We have discontinued the thickened edge on oil and use that material to make the wider mat. Instead of a 20 foot we have a 22 foot mat, because the quantity of material used in that thickened edge was equivalent to about one additional foot of width. It may be a bit early to pass any comments as to whether it is going to be successful, but we carry our 2½ inch thickness right to the shoulder line, instead of going to a 5 inch thickened edge, as we previously had.

One thing we can say in favor of the additional width is that I don't believe it will get quite so much wheel traffic right out on the edge, and with our shoulder-to-shoulder mat we get more stability underneath.

Mr. Perry:

I think it will be interesting to see the action of traffic out there on the decreased thickness of the mat.

R. J. Templeton:
Wyoming

In southwestern Wyoming about 6 years ago, we laid I think somewhere near 120 miles of 2½ inch mat, using SC-3 oil, and we used a thickened edge. These mats were constructed only 20 feet in width. On my trip over here I had an opportunity to look at the edges. I think they are all right. That is, where you have a narrow mat. It certainly stops the trouble you have with your feather edge. A feather edge section would ravel, and these thickened edges have stood up wonderfully well.

Mr. Perry:

Have you had any noticeable effects of moisture accumulated in the thickened edge?

R. J. Templeton:
Wyoming

I didn't notice any. These were not trench sections; just thickened edges.

Mr. Perry:

Mr. Marsh of Kansas, do you have something to say?

L. L. Marsh:
Kansas

Yes. About 8 years ago we constructed several heavy SC mats with thickened edges; that is, trenched, and those were built on a grade approximately 32 feet wide. The mat, however, was about 24. We didn't have any trouble and, in the natural course of events, we forgot to thicken them. At the present time, we can see that those thickened edges were a great advantage, especially where you had a 6 inch crown and heavy trucks. We haven't noticed where that trench has collected or held any water.

H. G. Haight: Idaho

I have noticed one fault with the thickened edge in Idaho, and that is, due to the extra compaction you get in the thickened edge section, a valley or trench is caused along the edge or shoulder of your mat. When it rains, the water sheds off to near the edge of the mat and runs into this little valley, which is about ½ inch deep. That causes stripping and early cracking of the mat over the top of the thickened edge. That is due, I think to the additional thickness of the material which compacts under traffic and forms a depression.

Mr. Perry:

I believe Mr. Worthington in his paper brought out something in regard to the thickened edge. In a nutshell, they found it necessary to place drains at the low points of the grade to take out accumulated moisture in this trench type of construction.

I still have a long list of questions, but it is now 5 o'clock and we are going to adjourn very shortly. I want to take this occasion to thank each and all of you for your splendid attention and cooperation this afternoon.

Meeting is adjourned.

Papers Prepared

on

Maintenance of Bituminous Surfaces

for

The Montana
National Bituminous
Conference

Maintenance of Bituminous Surfaces

By

R. J. TEMPLETON
Maintenance Engineer
Wyoming State Highway Department

The ever increasing problem of maintenance was largely if not entirely the deciding factor resulting in the reorganization of the Wyoming State Highway Department, which took place on July 1st of this year. Since its creation over two decades ago, the Department had functioned with but few changes taking place in the original organization set-up.

CONSTRUCTION AND MAINTENANCE ORGANIZATION

For the purpose of administration and supervision on both construction and maintenance, the State, which covers an area of 97,914 square miles, is divided into ten Residencies, each with a Resident Engineer in charge. Resident Engineer headquarters are located at Cheyenne, Laramie, Rawlins, Rock Springs, Kemmerer, Casper, Lander, Basin, Sheridan and Newcastle. All construction activities and maintenance operations are supervised by two Construction Engineers and a Maintenance Engineer respectively; they are directly responsible to the Superintendent. Resident Engineers are in charge of all construction and maintenance work within their respective territories; they are accountable to both Construction and Maintenance Engineers.

UNIT MAINTENANCE ORGANIZATION

To provide for efficient maintenance operations in all sections of the State throughout the entire year, each Residency is divided into two or more unit areas. Usually the unit area organization consists of about five men with a Crew Foreman in charge. With few exceptions, unit headquarters are located in the larger towns, most frequently County Seats. The maximum allowance of regular maintenance employees assigned to each Residency is based on the size of residency, mileage, volume of traffic, etc. The total for the State is set at 150 exclusive of shop mechanics, 11 of whom are regularly employed. During the summer months and periods of emergency, the regular force is augmented as required, sometimes more than doubled, with help employed on a temporary or seasonal basis. Boundary lines of Residencies, mileage maintained and allocation of regular maintenance employees are shown on the accompanying map.

BITUMINOUS SURFACE MAINTENANCE EQUIPMENT

The Highway Department is well supplied with equipment necessary in the maintenance of all types of Bituminous surfaces within the State.

This equipment consists of 22 Diesel Powered Motor Graders, practically all with scarifier attachments; 11 Gas Powered Motor Graders; 48 Tanks ranging from 500 to 2,200 gallon capacity, most of which are insulated, all mounted on trucks; 16 Pull-type Asphalt Distributors; 3 Power Brooms; 13 Sand and Chip Spreaders; 2, 8-ton Three-Wheel Rollers, 1, 5-ton Tandem Roller; 10 Pull-type "Texas" Rollers; 4 Disc Planers; 20 Motor Spray Asphalt Distributors; 27 Hand Spray Asphalt Distributors commonly called "Tar Kettles" and 171 Trucks from 1½ to 5 ton rated capacity. Most of this equipment is in daily use during the summer months. Power shovels are frequently used in the production of sand and gravel required on maintenance. Five are owned and operated by the Department.

LIQUID ASPHALT MATERIALS AND STORAGE

Wyoming is the principal oil producing State in the Intermountain Area. Liquid asphalt materials are produced at refineries located at Casper, Cody, Lovell, Thermopolis and Greybull. Shipments are made by tank car or trucks depending on haul distance. Nine State owned tanks, ranging from 3500 to 20,000 gallons in capacity, located at advantageous points throughout the State, provide ample storage facilities for RC cutbacks used in the repair and sealing of Bituminous surfaces. Several additional tank installations are to be made this year.

MAINTENANCE SHOPS AND DEPOTS

Well equipped shops are located at the headquarters of each residency and improved facilities for servicing and housing of equipment are provided for in most of the unit areas. The State now has 11 modern Shops and Depots of one story brick, concrete and steel construction, and two more are to be constructed this year. Shop mechanics make frequent visits to the Depots to check, adjust and make repairs on road machinery being used in that vicinity. The personnel of the unit organizations attend to the regular servicing of equipment used by them. The economical operation of road equipment is largely dependent on its proper care, and the importance of adequate housing facilities cannot be overstressed.

MAINTENANCE COSTS

During the Fiscal year ending March 31, 1939, Wyoming spent the sum of $856,939.31 maintaining its State Highway System of 3,773 miles, averaging slightly over $227 per mile. Approximate amounts and percentages per mile, cover the following described operations:

Surface Maintenance, Ordinary and Periodic Combined	$137 per Mile or 60%
Snow Control and Removal, Including Sanding	$ 39 per Mile or 17%
All Other Maintenance Operations	$ 51 per Mile or 23%

The accompanying Recapitulation of maintenance expenditures shows amounts spent in each of the 23 Counties.

MILEAGE AND SURFACE TYPES

Prior to this year's construction, Wyoming's State Highway System of 3,773 miles consisted of the following surface types:

Bituminous Surface — All Types	3,089 Miles
Concrete Surface	28 Miles
Gravel Surface	331 Miles
Earth Surface	325 Miles
Total	3,773 Miles

The Bituminous surface types range from 3 inch plant mix using SC-4 road oil, and 1½ inch road mix using both SC-3 road oil and MC-3 cutback, to penetration with MC-1 cutback followed up with sand or chip seal using RC-2 or RC-4 cutback. A considerable mileage of the SC-3 mats were built previous to 1934 on grades with insufficient base material. With increased traffic and heavier loads, maintenance costs are high, and on some sections where excessive, reconstruction seems to be the only remedy. Base and subgrade failures are responsible for over 90% of all surface maintenance. On our recent construction jobs, where soils in the subgrade were compacted to required density and base courses stabilized, surface maintenance costs are exceptionally light.

A. **Shoulder Maintenance.** The breaking down of the edges of bituminous surfaces is invariably due to lack of shoulder support. This is plainly evident on many hundreds of miles of old low cost type roads in Wyoming. To remedy this condition it is usually necessary to stabilize the shoulders, also flatten embankment slopes to obtain needed lateral support; only gravel or the better grade soils should be used for this purpose. Flat or high shoulders invite saturation resulting in weakened stability. It is the general practice in Wyoming to keep shoulders trimmed down and sloped to permit surface water to flow rapidly to the ditches. We have done a considerable amount of shoulder sealing with SC oils, and MC and RC cutbacks; the results were usually satisfactory. Thus far no attempt has been made to stabilize shoulders using Bituminous materials, however, we have widened some of the mats eliminating the shoulders entirely.

B. **Surface Repairs (Sealing).** Seal treatments in Wyoming are done almost exclusively with the RC cutbacks. Unless surface texture shows coarseness sufficient to prevent skidding, a cover coat of sand or stone chips is deemed necessary. For Sand Seal we use .12 gallon of RC-2 cutback and from 5 to 10 pounds of sand per square yard. For Stone Chip Seal .25 to .30 gallon of RC-4 cutback and 20 pounds of chips per square yard.

Stone Chips are required to meet with the following specifications.

''Stone chips shall be sound, durable crushed ledge rock, or boulders, or clean pea gravel, and shall conform to the following requirements as to grading when tested by laboratory sieve with square openings'':

	Opening in Inches	Per Cent
Passing ¾ inch sieve	.750	100
Passing ½ inch sieve	.500	95-100
Passing No. 10 sieve	.0787	0-5

It is the general practice in Wyoming to scarify and reprocess with additional SC road oil or cutback, in the amount required, mats which are dried out and badly cracked. Apparently oxidation or absorption have taken place causing disintegration; patching is an endless job in a case of this kind and results obtained from sealing are only temporary at best.

C. In Northern Wyoming it has been necessary to stabilize the base course on many miles of old road, and then relay the original mat material. This work was done in the following manner. The mat was torn up (scarified) and laid to one side of road, the opposite half was then shaped and gravel with proper amount of suitable binder hauled in and spread, watered and thoroughly rolled after which the windrow of mat material was shifted and the other half treated in the same manner. Mat material was thoroughly broken down, proper amount of asphalt oil added, reprocessed and laid. In places where impossible to provide detour, Flagmen were used to direct traffic over the road.

D. **Winter Maintenance.** During the winter months, only a small amount of surface repair work is carried on; this consists mostly of skin patching with RC-2 cutback and stone chips, and a limited amount of sealing during periods of favorable dry weather. Most of the time maintenance crews are busily engaged on important work of an entirely different nature—Snow Control and Removal. Besides keeping the roads open and safe for traffic, snow removal is of unquestionable value in the maintenance of Bituminous surfaces as snow allowed to melt on the shoulders is liable to saturate and weaken the base and cause serious trouble.

CONCLUDING REMARKS

As previously mentioned, base and subgrade failures are the cause of over 90% of all surface maintenance in Wyoming. Subgrade failures are usually difficult and somewhat expensive to correct. Any one or combination of the following conditions may cause plenty of trouble:

Unsatisfactory soil.
Lack of soil compaction resulting in settlements.
Poor drainage causing saturation of subgrade, capillary and frost boils.
Lack of base course stability.

All can be remedied by well known methods.

Surface failures are indicated by raveling, corrugating or cracking. Raveling, if caused by lean mix or oxidation of Bitumen, can usually be corrected by sand or stone chip seal. Sometimes it is necessary to remix with additional Bitumen. If moisture is the cause, it is necessary to tear up, dry out and remix.

Corrugations are corrected by ripping up and remixing. In case water is the cause, it is necessary to dry the mat material and base. Where excess oil is the cause, aeration or additional aggregate is generally required.

To correct cracking, it is usually necessary to tear up and remix the mat and stabilize base. Cracking, if light, can be remedied at least temporarily by sealing with sand or stone chips.

MAINTENANCE EXPENDITURES

April 1, 1938 to March 31, 1939

		DEBITS											CREDITS		
Date	Reference	M1	M2	M3	M4	M5	M6	M7	M8	M9	M10	Tot. Debits To Date	Amount	Tot. Credits To Date	Bal.
		Surface, Routine	Surface, Periodic	Shoulders & Side Approaches	Roadside & Drainage	Traffic Services	Traffic Lane & Guide Line Painting	Snow, Ice & Sand Control	Snow, Ice & Sand Removal	Structures	Extraordinary (Special)				
RECAPITULATION															
County															
Albany		20,162.21	11,304.34	750.14	2,918.90	390.80	2,321.72	1,684.70	11,444.14	218.83		51,195.78	51,195.78		0
Big Horn		10,267.56	9,252.88	1,246.02	6,163.18	650.00	1,675.65	268.23	3,126.80	957.05	236.21	33,843.58	33,843.58		0
Campbell		12,656.73	8,859.43	1,139.86	2,948.81	196.40	619.18	465.88	2,387.49	338.22	10.68	29,622.68	29,622.68		0
Carbon		24,447.93	23,960.55	1,316.00	4,314.94	1,248.10	2,416.76	2,008.51	10,508.40	809.64	694.76	71,725.59	71,725.59		0
Converse		6,483.51	3,954.24	1,893.64	1,820.38	251.73	1,485.03	98.23	732.40	746.04	1,167.24	18,632.44	18,632.44		0
Crook	Crook	12,946.11	1,117.19	956.63	3,017.87	437.32	939.41	1,268.95	4.457.46	402.87	29.24	25,573.05	25,573.05		0
Fremont		21,958.04	19,675.71	3,493.41	9,537.73	854.72	2,916.52	1,130.53	7,734.20	1,417.81	8.01	68,726.68	68,726.68		0
Goshen		5,055.74	2,765.90	2,956.33	2,740.87	624.69	1,121.39	70.32	605.17	380.03	809.33	17,129.77	17,129.77		0
Hot Springs		2,120.23	426.74	238.81	2,604.88	195.52	374.97	22.54	1,550.43	474.28		8,008.40	8,008.40		0
Johnson		15,669.96	9,542.40	1,336.02	3,995.95	332.61	1.140.48	1,093.45	4,506.13	659.88	392.68	38,669.56	38,669.56		0
Laramie		11,600.47	12,962.28	1,138.92	3,596.76	1,137.30	2,003.81	2,558.74	3,155.05	48.46	745.58	38,947.37	38,947.37		0
Lincoln		16,665.91	7,957.38	335.72	5,980.33	1,762.74	1,330.28	283.89	13,198.49	1.837.66	855.19	50,207.59	50,207.59		0
Natrona		21,133.29	13,154.06	3,431.97	5,706.94	1,267.15	2,313.29	643.18	5,442.61	752.38	23.55	53,868.42	53,868.42		0
Niobrara		9,093.08	11,812.00	946.64	1,588.81	287.24	1,481.42	211.47	1,638.26	317.42	17.91	27,394.25	27,394.25		0
Park		3,744.10	6,419.28	105.73	8,140.72	397.66	1,173.90	6.10	1,681.58	619.02		22,288.09	22,288.09		0
Platte		8,162.71	6,090.64	1,213.16	3,677.91	821.37	1,627.22	184.13	1,496.99	382.00	364.31	24,020.44	24,020.44		0
Sheridan		14,291.84	17,206.34	402.78	7,219.72	638.30	1,421.37	403.05	5,636.95	968.27	1,133.20	49,321.82	49,321.82		0
Sublette		14,345.43	7,073.94	200.55	4,109.41	484.94	746.05	1,037.75	11,157.93	698.08	1,066.80	40,920.88	40,920.88		0
Sweetwater		30,119.76	19,122.76	1,388.68	4,170.83	998.99	2,619.16	386.65	4,403.04	343.37	432.82	63,986.06	63,986.06		0
Teton		9,902.04		125.84	5,243.49	477.48	547.96	529.82	26,443.96	106.17		43,376.76	43,376.76		0
Uinta		9,996.18	7,771.77	1,059.30	2,285.25	603.53	1,101.86	224.38	4,133.81	744.75	9.39	27,930.22	27,930.22		0
Washakie		4,996.50	1,294.58	93.18	3,382.67	193.66	525.47	43.96	1,907.27	286.04		12,723.33	12,723.33		0
Weston		11.935.93	16,831.36	110.14	1,907.46	211.99	876.59	767.30	5,681.05	126.72	378.01	38,826.55	38,826.55		
TOTAL		297,755.26	218,555.77	25,879.47	97,073.81	14,464.24	32,779.49	15,391.76	133,029.61	13.634.99	8,374.91	856,939.31	---	856,939.31	0
Per Cent of Total		34.74%	25.50%	3.02%	11.33%	1.69%	3.83%	1.80%	15.52%	1.59%	0.98%	100.00%			

STATE OF

WYOMING

MAP SHOWING STATE HIGHWAY SYSTEM AND INCORPORATED TOWNS

Legend

County Seats

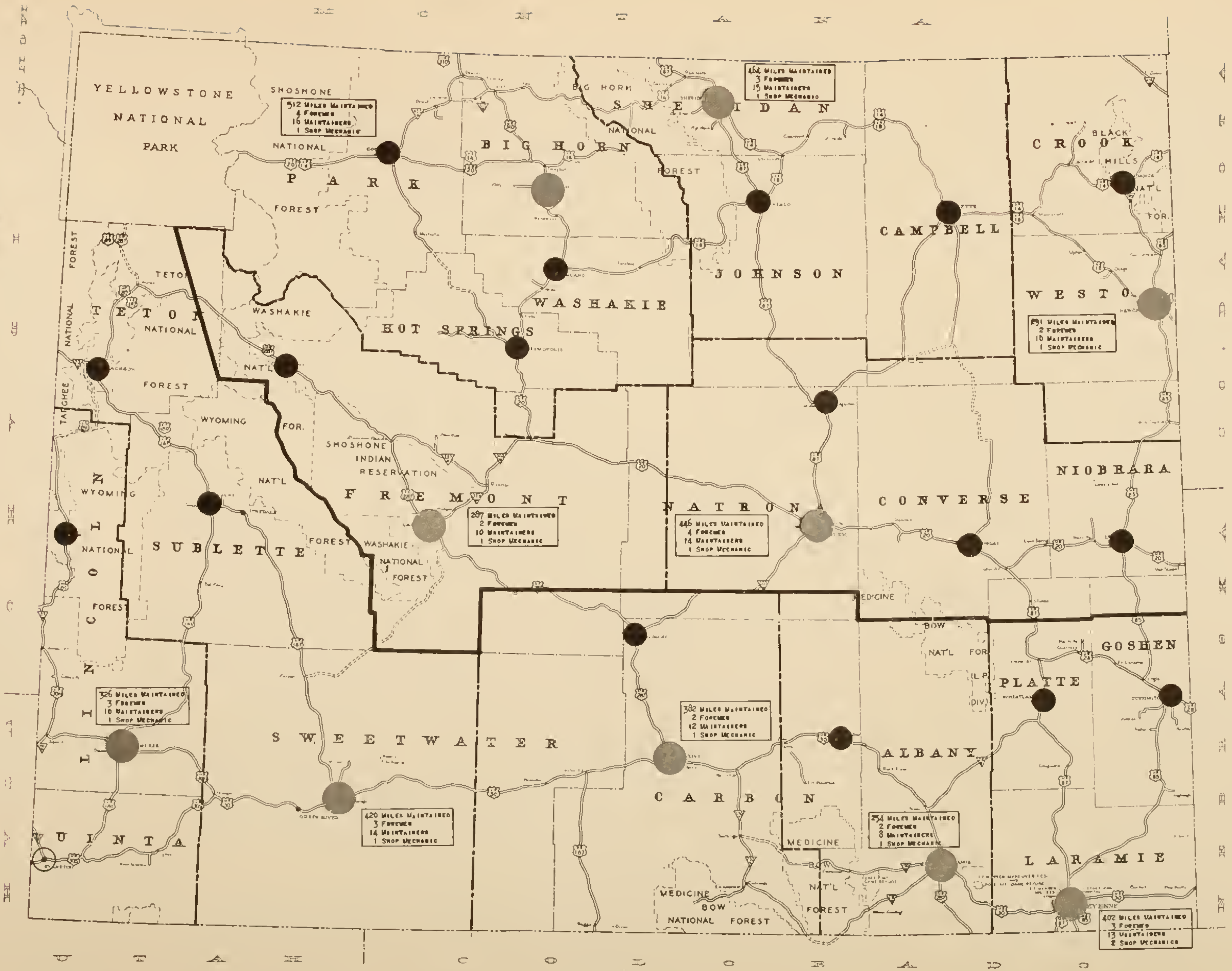
Incorporated

Unincorporated

L-E-G-E-N-D

Headquarters of Residency ●

Headquarters of Unit Area ●



MONTANA
IDAHO
SOUTH DAKOTA
NEBRASKA
UTAH
COLORADO
YELLOWSTONE NATIONAL PARK
SHOSHONE NATIONAL FOREST
PARK
BIG HORN
BIG HORN NATIONAL FOREST
SHERIDAN
CROOK
BLACK HILLS NAT'L FOR.
CAMPBELL
JOHNSON
WASHAKIE
HOT SPRINGS
WESTON
TETON
TETON NATIONAL FOREST
TARGHEE
WASHAKIE NAT'L FOREST
WYOMING NAT'L FOR.
SHOSHONE INDIAN RESERVATION
FREMONT
NATRONA
CONVERSE
NIOBRARA
LINCOLN
WYOMING NATIONAL FOREST
SUBLETTE
WASHAKIE NATIONAL FOREST
MEDICINE BOW NAT'L FOR. (L.P. DIV.)
PLATTE
GOSHEN
SWEETWATER
CARBON
ALBANY
LARAMIE
UINTA
MEDICINE BOW NATIONAL FOREST
512 Miles Maintained
4 Foremen
16 Maintainers
1 Shop Mechanic
464 Miles Maintained
3 Foremen
15 Maintainers
1 Shop Mechanic
291 Miles Maintained
2 Foremen
10 Maintainers
1 Shop Mechanic
287 Miles Maintained
2 Foremen
10 Maintainers
1 Shop Mechanic
446 Miles Maintained
4 Foremen
14 Maintainers
1 Shop Mechanic
326 Miles Maintained
3 Foremen
10 Maintainers
1 Shop Mechanic
420 Miles Maintained
3 Foremen
14 Maintainers
1 Shop Mechanic
382 Miles Maintained
2 Foremen
12 Maintainers
1 Shop Mechanic
234 Miles Maintained
2 Foremen
8 Maintainers
1 Shop Mechanic
402 Miles Maintained
3 Foremen
13 Maintainers
2 Shop Mechanics
Green River
Wheatland
Cheyenne

Maintenance Practices on Bituminous Pavements In Colorado

By

CHAS. E. SHUMATE
Asst. Supt. of Maintenance
Colorado State Highway Department

During the past two years in Colorado we have been experimenting with different types of shoulder treatment to prevent water from entering under the bituminous mat from the shoulder. So far, our experience indicates that the best solution is to construct an asphaltic membrane adjacent to the pavement. All loose shoulder material is bladed back from the edge of the mat for a distance of at least 2 feet then a single application of approximately three tenths of a gallon of MC is applied on the area cleaned. As soon as this material has been blotted up the outside edge (about 1 foot) of the mat edge and about one foot of the area treated with MC is given an application of about ¼ gallon of RC-2 and the loose gravel is then bladed back to its original position. So far it appears that this method is proving successful. However, on new mats being constructed the tack coat is being applied to the base about four feet wider than the area to be covered with the new mat.

In the past year Colorado has placed a bituminous mat 48″ wide along both sides of some 150 miles of narrow Portland Cement Concrete pavements principally for the purpose of pavement widening and to reduce shoulder maintenance costs along these pavements. We use a trenching machine especially designed for this work, excavating along the pavement to a depth of 2½ inches four feet in width. This space is then filled with premixed bituminous material, bladed to uniformity and rolled, later it is sealed.

We have used both open and close graded mixtures in this work with SC and MC oils as binders. Results so far have been excellent and while the cost has been approximately $1,500.00 per mile we feel that it has been justified.

Our surface maintenance like all other states presents many special problems. When the failures from faulty subgrade are not too numerous we dig out the area affected to a depth of at least 12 inches, backfill with stable material well compacted then replace the surface area with premixed material of the same depth as the original mat. We have found on some sections where there is considerable checking of the mat due to faulty subgrade, but where the failure has not caused displacement, that a heavy application of cutback applied to the area and covered with chips has in many cases stopped the area from giving any more trouble, inasmuch as the heavy seal has prevented surface moisture from entering the base from the surface. In areas where the surface failure is extensive due to faulty base, we remove the bituminous material completely off the road, scarifying and blading it into windrows where it is loaded and hauled to nearby stock piles. The faulty base is then corrected, the bituminous material is hauled back, retreated, and layed, using power graders. With a mat which has been constructed using SC oils this retreatment of the mat material is relatively simple, where MC or RC has been used in the original mat the operation is much more difficult, however, we have successfully retreated several mats where MC-3 had been used as a binder.

We have found in Colorado that the use of premixed material and the use of the small asphalt kettle equipped with motor drive pump and hand spray provide the best and most economical method of patching. Of course, when the areas to be repaired are large the use of motor graders, large distributors, etc., are used. Penetration patch work is done in the winter months, but during the summer we prefer to use premixed material for patches of any depth.

We have had considerable experience in the past few years in the resurfacing of small sections of old Portland Cement Concrete pavements. Most of the material used in this work has been an open graded mix with MC as the binder material, however, we have obtained excellent results using a close graded mix and SC-3 oil holding the quantity of oil quite low and the maximum size of aggregate to ¾ inch, this material requires a seal with cutback soon after laying, but it provides a material which can be stockpiled over long periods of time and still remain workable. Our method of doing this resurfacing is as follows:

The old pavement is broomed thoroughly, then an application of about one-tenth gallon of RC-2 is made, this is allowed to set until it becomes quite sticky, the premix is then placed and brought to a uniform surface by raking or blading, the patch is then rolled with a light roller. Later it is sealed with cutback and covered with stone chips. In doing this work only one half of the pavement is worked at a time giving the traffic access to the other half.

We have several patches of this type which have been under traffic of four to five thousand vehicles per day for several years and which are still giving excellent service.

Question I would like to have asked:

What experience has anyone had in subgrade stabilization with bituminous materials in maintenance work? Its cost, results, etc.

Retreatment of Bituminous Surfaces

By

W. K. BECKHAM
Maintenance Engineer
South Carolina State Highway Department

The South Carolina State Highway System contains 4,000 miles of bituminous surfacing of which 3,500 miles are of low-cost type commonly known as "surface treatment." This type consists of a thin mat about three-fourths of an inch thick placed on a sand-clay, topsoil, or stabilized base. The mat is simply a wearing and protective coat for the base which carries the load. If this protective coat, known as "surface treatment" is worn through to the base or if the asphalt becomes brittle or cracks, water penetrates the mat and goes into the base resulting in serious damage to the road. To protect properly the investment in the road, the surface should be treated when it shows that its thickness has been materially reduced by wear or when the asphalt shows the first sign of brittleness. Experience has shown that it is necessary to retreat the surface about every five years to enliven and reseal the mat and to retain a smooth non-skid riding surface. If the retreatment is unduly delayed and the base as a whole is damaged, it will be necessary to reconstruct the entire section which will usually cost two or three times as much as a retreatment.

It is the Highway Engineer's duty to see that satisfactory highway service is rendered at the lowest possible cost. In developing the process of retreatments during the past twelve (12) years every effort has been made to carry out this policy. The mixed-in-place retreatment not only has been the cheapest but has also given satisfaction on at least 95% of our work. Pre-mixed material has been used on the remaining 5% which is in cities. This material usually costs about twice as much per ton as mixed-in-place material, but it is more desirable on municipal work where the streets must be kept open to traffic.

There are about 3,500 miles of surface treatment in our system today compared to 1,200 miles five years ago. This increase in mileage has resulted in the volume of retreatment increasing steadily.

In South Carolina the work of retreatment is done entirely by the maintenance organization. In each of the four districts we have what are known as "floating units" which are crews of men and equipment specialized in retreatment work. Each of these four crews are under a Superintendent who is responsible to the District Engineer. As nearly as possible the same crew is retained throughout the season and follows the work from county to county. The key men of the organization are retained during the winter months on regular maintenance and betterments. In this manner an experienced crew is available every spring to start retreatment work.

The retreatment of bituminous surfaces is essentially the application of 50 pounds of ⅝″ aggregate mixed-in-place with approximately 0.6 gallon rapid curing asphalt. This quantity is varied at times on sections requiring special attention. Ordinarily, on particularly rough sections the quantity is increased. The aggregate is a combination of 25 pounds of:

Aggregate No. 8

% Passing (By Weight)	Minimum	Maximum
⅝″ Square Screen	100.0	
½″ Square Screen	97.0	100.0
⅜″ Square Screen	75.0	100.0
No. 4 Mesh Sieve	20.0	60.0
No. 16 Mesh Sieve	0.0	5.0
No. 100 Mesh Sieve	0.0	1.0

and 25 pounds of:

Aggregate No. 11

% Passing (By Weight)	Minimum	Maximum
⅜″ Square Screen	100.0	
No. 4 Mesh Sieve	90.0	100.0
No. 16 Mesh Sieve	50.0	85.0
No. 50 Mesh Sieve	2.0	20.0
No. 100 Mesh Sieve	0.0	5.0
Removed by Elutriation	0.0	3.0

These aggregates have a maximum Los Angeles abrasion loss of 40; however, a loss as high as 65 has been permitted in certain cases where there is a sizable saving in bids due to low freight rates. Recently a new grading of aggregates was tried which is really a little finer than the combination of the above two. The specification on this material is:

Aggregate No. 15

	Minimum	Maximum
Passing ⅜″ Square Mesh	100%	
Passing No. 4	70%	95%
Passing No. 8	35%	60%
Passing No. 16	10%	25%
Passing No. 100	0%	3%

This aggregate has proved to be good, making a dense waterproof mat and has the added advantage of having the crew handle only one material instead of two. During the present season an 80 pound retreatment was used on one section in which only Aggregate No. 11 was used. This type of material is available locally and if it can be used, will result in a large saving.

It has been found in handling the aggregate that it increases the progress of the retreatment crew to stockpile the material at two or three mile intervals along the project on which the work is in progress.

The asphalt used in retreatment work is rapid curing cutback asphalt meeting the A.A.S.H.O. Specifications. The viscosity of the material used varies from 80 (Saybolt-Furol) to 350, depending on the air temperature. The asphalt is heated to 120° F. and applied at the rate of 0.6 to 0.7 gallon per square yard depending on the type of aggregate and the condition of the old surface.

No special equipment is used in retreatment work other than a combination mixing and finishing machine which has been developed in South Carolina. This machine is essentially two long wheel-base multiple blade maintainers connected together. A single drawbar is provided so that a large tractor is capable of pulling both units. A strike-off blade covering the full width of the road is mounted behind the maintainers. This blade is adjustable from 18′ to 24′ wide and is made so that the crown in the blade may be varied to suit road conditions. In order to obtain well aligned edges the machine has a steering rig which enables the operator to closely follow the cord line previously placed along the edge of the road. Other pieces of equipment necessary in the retreatment work are a loader or crane, a fleet of 1½ ton trucks, a 600 gallon distributor, a small grader, a large tractor, a large multiple blade maintainer, a boiler or booster, and a 5-ton Tandem roller.

Prior to the season for retreatment the maintenance superintendents in the various counties in which retreatment work is proposed do all necessary patching which consists of building up all low places and making all necessary repairs in the surface and base. The finishing machine, due to its long wheelbase, is capable of taking care of any small irregularities in the surface, but it is necessary to make all major repairs ahead of the retreatment work.

The operation of actually placing the retreatment begins with a survey party re-running the original centerline of the section to be retreated. A cord line is set along the edge of the roadway and is valuable in maintaining neat, well aligned edges in the work. A crew of men with brooms or a road sweeper clean the roadway before placing a "tack coat" of the RC cutback asphalt of 0.12 - 0.18 gallon per square yard. The aggregate is spread uniformly over this "tack coat" by spreaders mounted on the trucks. In spreading the aggregate a ½ foot width on either edge of the roadway is left uncovered. This is to prevent loss of material in the mixing operation. It is often necessary to air dry the aggregate. This is done by shifting the aggregate back and forth over the roadway with a small grader or with a multiple blade road maintainer. The remaining asphalt necessary to bring the total to 0.6 or 0.7 gallon per square yard is applied on the dried material. The mixing operation consists of making one and one-half round trips with a large multiple blade maintainer, that is a trip down one side of the roadway, back up the opposite side and down through the middle. In some cases it is necessary to make additional trips, but ordinarily one and one-half are enough. This operation leaves the material partially mixed and well distributed over the roadway. This is followed immediately by the finishing machine which travels continuously in one direction and leaves the material completely mixed and finished in place. This machine eliminates all seams, ''turn-around'' places and practically all hand work. The mix is rolled with a 5-ton Tandem roller beginning at the outer edge of the roadway and working toward the center. After the finished material has been rolled, it is allowed to cure for two or three days before opening the road to traffic. In some cases where it is impractical to close the road, the retreatment has been covered with 5 to 10 pounds of sand and opened to traffic immediately.

Using approximately ten trucks, one distributor and one finishing machine the retreatment can be applied to the roadway at the rate of approximately 3,000 square yard per hour.

The actual cost of placing the retreatment consisting of 50 pounds of aggregate and 0.6 to 0.7 gallon per square yard of asphalt is approximately 12 cents per square yard. Prorating this over a five year period makes the annual cost 2.4 cents per square yard per year. This retreatment cost plus 0.7 cents per square yard per year for routine surface maintenance, makes the total cost of maintaining the surface treatments approximately 3.1 cents per square yard per year.

About twelve years ago retreatments were made for the purpose of resealing and thickening the mat. Retreatments for that purpose were of the inverted penetration and the broom drag types. These retreatments did not materially improve the smoothness or riding quality of the old roads, which were becoming rougher each year. The mixed-in-place retreatment, described in this paper, not only reseals and adds thickness to the old mat but also improves the smoothness of the bituminous surfaces. With proper maintenance and the application of such a retreatment approximately every five years, the bituminous surfaces of the low cost type should be kept in good condition for an indefinite time.

Maintenance of Bituminous Surfaces In West Virginia

By

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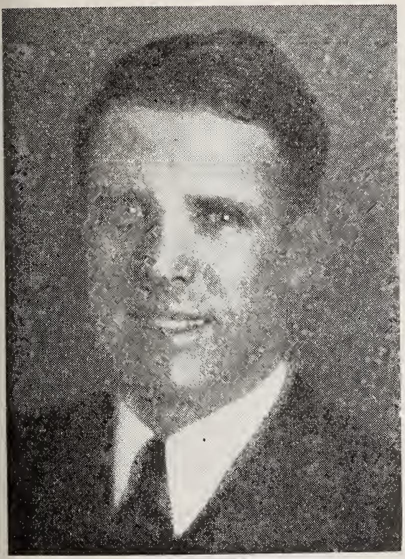

SURFACE REPAIRS

1. Seal Treatments

We have been successful to a marked degree in the use of light seal treatments on bituminous surfaces that are smooth or reasonably so, and have sufficient structural strength to carry the load for which they are designed.

These light seals should be applied when the pavement surface has weathered and dried to a gray color, when cracks and checks appear, or when the surface begins to ravel and pick off under traffic.

The purpose of these seal treatments is as follows: (1) To prevent the intrusion of surface water with its resultant tendency to start disintegration. (2) To prevent the intrusion of air with its tendency to accelerate oxidation and evaporation, which causes the bituminous material to become brittle and break up. (3) To liven up and make more flexible the existing surface by the penetration of the new oil into the old dry and hard surface. (4) To cover the existing surface with a fine grained sandpaper like finish composed of a finely graded aggregate, which furnishes the skid resistance which is so desirable. (5) To eliminate the surface voids which hold the surface water. During the winter season the alternate freezing and thawing of the small volumes of water in these voids between the particles of aggregate causes them to loosen and whip off under traffic. At other times these voids create small vacuums under the tires, and the suction formed has a tendency to pull the aggregate particles loose and thus start ravelling.

The types of bituminous material recommended for these seal treatments are either tar or asphalt emulsion, and the reason for recommending these two materials for these light seal treatments is because they have a better penetrating quality and have been found to offer better skid resistance. The rates of application recommended for the bituminous material are from 0.20 to 0.25 gal. per square yard, depending upon the condition of the existing surface. For cracked "alligatored" surfaces or all surfaces in cool weather use Tar RT-6 or RT-7, Asphalt Emulsion AE-2 or AE-3. For smoother, tighter surfaces or all surfaces in hot weather use Tar RT-8 or RT-9, Asphalt Emulsion AE-3.

The types of aggregate recommended are crushed limestone, crushed slag or other similar materials. The rates of application of the aggregate are from 8 to 10 pounds per square yard. The gradation of the aggregate is a graded material with 100% passing a 3/8″ square opening and not over 2% passing a 100 mesh.

The sequence of operations to use in applying the light seal treatment is as follows:

1. Clean and sweep.
2. Apply 0.20 to 0.25 gal. per sq. yd. bituminous material.
3. Spread immediately 8 to 10 pounds per sq. yd. No. 12-a Aggregate. The aggregate to be spread with approved mechanical spreader. Back the truck in order to spread the cover coat on the bituminous material ahead of the truck tires. This eliminates picking up or tracking the bituminous material.
4. Roll lightly, immediately, with 7 to 8 ton roller. (Note): Application of bituminous material and aggregate to be made on one-half of pavement width at one time.

The periodic application of these light seal treatments will eliminate almost entirely the patching of holes, because it is a hole or break preventative treatment. All surface holes or breaks have to start small and will not begin at all if the proper balance is maintained between live bituminous material and aggregate.

It has been repeatedly said that most surface failures are due to the failure of the base. My contention is that most of the base failures which cause the surface failures are started by minor failures of the surface such as cracking, checking, etc. which allow the surface water to get into the pavement and base and thence to the subgrade. The stability of the subgrade, being impaired by the moisture, allows the base to move which in turn breaks up the surface. Therefore, if the surface is kept impervious to water and air by proper sealing, the possibility of base failure is minimized. This does not, of course, apply to the base failures resulting from poor subgrade or bad sub-drainage conditions. Their elimination is an entirely different matter.

An earth road is perfectly solid as long as it remains dry, but moisten it and an entirely different condition exists. The earth subgrade under a macadam base and surface is entirely adequate to support the load as long as it remains dry, which it will do as long as there is a waterproof surface over it, but permit the surface or roof over it to leak, then in its moistened and plastic condition its bearing power is materially reduced. Furthermore, as the earth subgrade becomes soft, the base aggregate is pushed down into it, and the soft earth displaced is pushed up between the particles of the base, making as a whole an unstable mass which offers such poor support to the bituminous surface course that it must necessarily ravel and break up under traffic loads, thus forming either an uneven mass of broken surface or pot holes.

Therefore, inasmuch as a large percentage of base failures with subsequent surface failures can be attributed directly to water and air getting into and through the surface of the pavement, we may well spend more time and money on our surface seals.

2. Light Surface Treatments

(25 lbs., 55 lbs., and 90 lbs.)

In studying the set-up and sequence of operations for these three light surface treatments, you will note that a number of different sizes of aggregates are used in their application, and also that there are quite a number of applications of both the bituminous materials and the aggregate.

The purpose of this is to build up a tight mechanically bonded surface mat by thoroughly filling all the voids from large to small with the proper combination of aggregate and bitumen.

The proper structural strength can be built into the surface course by filling the voids either with bituminous material alone or with a combination of the two. The former instance takes more bituminous material, which costs approximately five times as much as the aggregate; furthermore, the excess of bitumen has a tendency to become soft in hot weather with the resulting bleeding and shoving. It also is prone to become slippery in wet and frosty weather. Whereas, by using a combination of graded aggregate in successive applications with just enough bituminous material with each application to insure the proper bond, a watertight, aggregate, locked, surface mat will be obtained and there will be no danger of its being over-bituminized. Furthermore, the fine aggregate which is used for the final seal gives a fine sandpaper like finish which is the most skid resistant, due to the lack of voids to hold the water which is the lubricating medium which makes the tires skid.

Care should be exercised not to use too much of the final seal coat aggregate; first, because the excess is thrown out on the shoulders and wasted, and second, because this excess of fine material has an abrasive effect as it is whipped across the surface which injures the light surface coating of bituminous material; furthermore, as this excess aggregate is kicked off it carries with it a certain amount of the bituminous material, thus wasting it.

As soon as the final rolling is completed on the 55 and 90 lb. treatments, an extra application of approximately 0.15 gal. per square yard of bituminous material should be applied to each edge of the surface in strips from 18″ to 24″ in width. This affords an added protection to these vulnerable parts of the surface and will do much toward eliminating the expensive edge patching. A cover coat is not necessary on these edge applications because enough loose material from the final application of aggregate will dust to the edges. This edge treatment is much more necessary on narrow surfaces than on wide surfaces.

The uses of these three light surface treatments are described as follows:

25 lb. Treatment: This type and weight of treatment should be used for practically the same purpose as the light seal treatment and is recommended when funds are available for a little heavier seal, and when the existing surface is a little more rough, open, and uneven. The more uneven places should be patched out with some leveling material before this treatment is applied.

The following set-up is recommended:

a. Bituminous Material.
   1. Amount: 0.35 to 0.45 gal. per sq. yd.
   2. Type: Tar, (RT-7, RT-8, or RT-9) asphalt emulsion, (AE-3), cutback asphalt (RC-2 or RC-3).

b. Aggregate.
   1. Amount: 25 lbs. per sq. yd.
   2. Type: Crushed limestone, crushed slag, crushed gravel (100% crushed and put through a soft stone eliminator), other like materials (crushed).

c. Sequence of Operations.
   1. Clean and sweep.
   2. Apply 0.10 to 0.15 gal. per sq. yd. bituminous material (Tack Coat—allow to penetrate, but not to dry, as aggregate course should be applied while the tack coat is still tacky or sticky). (The amount and grade of material for the tack coat should vary with the condition of the surface to be treated.)
   3. Spread 15 lbs. per sq. yd. No. 12 aggregate with mechanical spreader.
   4. Level with broom drag.
   5. Apply 0.15 to 0.20 gal. per sq. yd. bituminous material.
   6. Spread 5 lbs. per sq. yd. No. 12 aggregate with mechanical spreader.
   7. Roll until well compacted. (Allow new surface to dry for at least 48 hours, in order to prevent flushing of road surface with bituminous material which results from making applications so fast that the volatiles have no chance to evaporate.)
   8. Apply 0.10 gal. per sq. yd. bituminous material.
   9. Spread 5 lbs. per sq. yd. No. 12-a aggregate with mechanical spreader.
   10. Broom and roll to finish. (Use 7 to 10 ton roller.)

Notes:—

For sizes of aggregate see table at end of this section, "Graded Requirements for Coarse Aggregate."

Application of bituminous material and aggregate to be made on one-half of pavement width at one time.

If Tack coat makes the road surface dangerous for travel, apply about 3 lbs. per sq. yd. of the No. 12 aggregate, deducting this amount from the 15 lbs. set-up.

When this type of surface treatment is to be used on an unsurfaced stone, slag, or gravel base, it will be necessary to adequately prime and prepare the base before the treatment is applied. For this prime, use 0.30 to 0.50 gal. per sq. yd. Tar, RT-2, RT-3 or RT-4, depending on temperature and tightness of base. It is important to use the heaviest prime that will adequately penetrate the base.

55 lb. Treatment: This type and weight of treatment should be used when the surface of the existing pavement is dry, open, cracked, somewhat ravelled and more or less uneven. Also, a surface in which there is need of building up some additional structural strength.

Before this treatment is applied, the existing surface should be thoroughly patched and the more uneven places brought out with leveling material.

The following set-up is recommended:

a. Bituminous Material.
  1. Amount: 0.55 to 0.65 gal. per sq. yd.
  2. Type: Tar, (RT-7, RT-8, RT-9 or RT-10) or cutback asphalt, (RC-3 or RC-4).

b. Aggregate.
  1. Amount: 55 lbs. per sq. yd.
  2. Type: Crushed limestone, crushed slag, crushed gravel (100% crushed and put through a soft stone eliminator), other like materials (crushed).

c. Sequence of Operations.
  1. Clean and sweep.
  2. Apply 0.10 to 0.15 gal. per sq. yd. bituminous material (Tack Coat—allow to penetrate, but not to dry, as aggregate course should be applied while the tack coat is still tacky or sticky). (The amount and grade of material for the tack coat should vary with the condition of the surface to be treated.)
  3. Spread 40 lbs. per sq. yd. No. 10 aggregate.
  4. Apply 0.30 to 0.35 gal. per sq. yd. bituminous material.
  5. Mix with power grader and spread with standard drag; or, mix and spread with approved mixing machine.
  6. Tack Roll.
  7. Spread 5 to 7½ lbs. per sq. yd. No. 12 aggregate with mechanical spreader.
  8. Roll until well compacted. (Allow new surface to dry for at least 48 hours, in order to prevent flushing of road surface with bituminous material which results from making applications so fast that the volatiles have no chance to evaporate.)
  9. Apply 0.15 gal, per sq. yd. bituminous material.
  10. Spread 5 to 7½ lbs. per sq. yd. No. 12-a aggregate with mechanical spreader.
  11. Broom and roll to finish. (Use 7 to 10 ton roller.)

Notes:

For sizes of aggregate see table at end of this section, "Graded Requirements for Coarse Aggregate."

Application of bituminous material and aggregate to be made on one-half of pavement width at one time.

If tack coat makes the road surface dangerous for travel, apply about 3 lbs. per sq. yd. of the No. 10 aggregate, deducting this amount from the 40 lbs. set-up.

When this type of surface treatment is to be used on an unsurfaced stone, slag, or gravel base, it will be necessary to adequately prime and prepare the base before the treatment is applied. For this prime, use 0.30 to 0.50 gal. per sq. yd. Tar, RT-2, RT-3 or RT-4, depending on temperature and tightness of base. It is important to use the heaviest prime that will adequately penetrate the base.

90 lb. Treatment: This type and weight of treatment should be used when there is need of building additional structural strength into the existing pavement, and when the existing surface is dry, open, cracked, ravelled and uneven.

Before this treatment is applied, the existing surface should be thoroughly patched and the uneven places brought out with leveling material.

The following set-up is recommended:

a. Bituminous Material.
   1. Amount: 0.80 to 0.95 gal. per sq. yd.
   2. Type: Tar, (RT-8, RT-9, or RT-10) or cutback asphalt, (RC-3 or RC-4).

b. Aggregate.
   1. Amount: 90 lbs. per sq. yd.
   2. Type: Crushed limestone, crushed slag, crushed gravel (100% crushed and put through a soft stone eliminator), other like materials (crushed).

c. Sequence of Operations.
   1. Clean and Sweep.
   2. Apply 0.10 to 0.15 gal. per sq. yd. bituminous material (Tack Coat—allow to penetrate, but not to dry, as aggregate course should be applied while the tack coat is still tacky or sticky). (The amount and grade of material for the tack coat should vary with the condition of the surface to be treated.)
   3. Spread 70 lbs. per sq. yd. No. 7-a aggregate.
   4. Apply 0.50 to 0.55 gal. per sq. yd. bituminous material.
   5. Mix with power grader and spread with standard drag; or, mix and spread with approved mixing machine.
   6. Tack Roll.
   7. Spread 5 lbs. per sq. yd. No. 12 aggregate with mechanical spreader.

8. Roll until well compacted. (Allow new surface to dry for at least 48 hours, in order to prevent flushing of road surface with bituminous material which results from making applications so fast that the volatiles have no chance to evaporate.)
9. Spread 8 lbs. per sq. yd. No. 12 aggregate with mechanical spreader.
10. Level with broom drag.
11. Apply 0.20 to 0.25 gal. per sq. yd. bituminous material.
12. Spread 7 lbs. per sq. yd. No. 12-a aggregate with mechanical spreader.
13. Broom and roll to finish (Use 10 ton roller).

Notes:

For sizes of aggregate see table at end of this section, ''Graded Requirements for Coarse Aggregates.''

Application of bituminous material and aggregate to be made on one-half of pavement width at one time.

If tack coat makes the road surface dangerous for travel, apply about 3 lbs. per sq. yd. of the No. 7-a aggregate, deducting this amount from the 70 lbs. set-up.

When this type of surface treatment is to be used on an unsurfaced stone, slag, or gravel base, it will be necessary to adequately prime and prepare the base before the treatment is applied. For this prime, use 0.30 to 0.50 gal. per sq. yd. Tar, RT-2, RT-3 or RT-4, depending on the temperature and tightness of base. It is important to use the heaviest prime that will adequately penetrate the base.

GRADED REQUIREMENTS FOR COARSE AGGREGATES

Total Per Cent Passing (Square Openings)

St'd Size No.	Nominal Size	3½″	3″	2½″	2″	1½″	1″	¾″	½″	⅜″	No. 4	No. 8	No.100
1	1½″–3½″	90–100		25–60		0–15		0–5					
1a	1″–3″		100		35–75		0–15						
2	1″–2″			100	90–100	35–70	0–15		0–5				
26	#4–2″			100	95–100		35–70		10–30		0–5		
3	¾″–1½″				100	90–100	20–55	0–15		0–5			
37	#4–1½″				100	95–100		35–70		10–30	0–5		
4	0–1½″				100	95–100	65–90		45–70		35–50		
5	⅜″–1″					100	90–100	40–75	5–20	0–10	0–5		
6	#4–1″					100	90–100		25–60		0–10	0–5	
7	#4–¾″						100	90–100		20–65	0–10	0–5	
7a	#4–¾″						100	90–100		5–20	0–5		
8	#8–¾″						100	90–100		30–65	5–25	0–5	
9	0–¾″						100	90–100	55–85		35–60		
10	#4–½″							100	85–100	35–70	0–10		
11	#8–½″							100	90–100	40–75	5–25	0–5	
12	#8–⅜″								100	85–100	20–40	0–10	
12a	100–4									100	90–100	20–65	0–2
13	0–4									100	85–100		10–30

3. Maintenance Repair Methods for Bituminous Mats Which are Failing After Several Years of Service, Due Apparently to a Breakdown of the Mat Structure Itself Because of the Drying Out of the Mat Which Results in Cracking and Disintegration

Suggested treatment. First, apply at the rate of 0.20 to 0.25 gal. per sq. yd., depending upon the extent of the disintegration, light tar or light cutback asphalt, and cover immediately with an application of about 5 lbs. per sq. yd. of crushed aggregate, No. 12 (See table) and allow to dry thoroughly. Second, carefully patch all holes and depressions, being careful to use the minimum amount of bituminous material which will hold the patching aggregate together. The size of the aggregate should vary with the depth of the holes and depressions. Third, apply one of the three light surface treatments (25 lbs., 55 lbs., or 90 lbs.) described earlier in this paper. The weight of the treatment selected should depend upon the condition of the existing surface and the funds available.

4. Comparison of Cost, Serviceability and Influence of Weather Between Bituminous Premix and Asphaltic Penetration Methods of Patching Breakups of the Traveled Way.

The method and material used to patch holes or breaks should be determined by the type of pavement surface involved. For example, the holes in a low class, practically worn out road-mix surface should not be patched with high class pre-mixed material, whereas on the other hand a high class bituminous concrete surface should not be patched with poured patches.

By the same token, a surface which is inclined to be over-bituminized should be patched with premixed material, in which the bitumen can be controlled, whereas a dry surface should preferably be patched by the penetration method, because a little excess bitumen carried over the dry surface will help it. Generally speaking, a patch should not be of a higher class material than the surface which is being patched.

Concerning the influence of weather—the average poured patch is generally more impervious to water and air because it is usually sealed tighter.

Concerning the serviceability—either type of patch will give service in proportion to the quality of workmanship involved in its placing.

Concerning the comparison of cost—it is more economical as a whole to use poured or penetration patches. This is supported by the following facts:

1. The majority of the holes to be patched are from 2″ to 6″ in depth and when premixed material is used there is a tendency to fill the holes, regardless of the depth, with this costly material because of the convenience; whereas, the deeper holes should be filled practically to the finished surface with local knapped or crushed base stone and then sealed in, as a poured patch, using a minimum amount of higher priced aggregate and bituminous material.

2. The majority of the holes are in the older and poorer types of pavement and when holes in such a surface are filled with premixed material there is a tendency for the old pavement to break around the new patch of higher class material and quite frequently the result is two holes, one on either side of the newly filled hole, whereas when these holes are filled with local stone and then sealed with higher class chips and bituminous material this sealing is not confined to the area of the patch alone, but is extended out over the adjoining poorer class surface, thus livening the old surface and minimizing deterioration around the new patch.
3. When deeper holes are filled entirely with the small aggregate premixed top course material, there is a tendency for the patch to shove or roll during hot weather. It is true this fault can be eliminated by using large aggregate bottom course material in the holes first, but this necessitates two kinds of material with the necessity for added transportation and storage facilities. This inconvenience alone discourages its use to the point where the workmen use only the surface or top course material.
4. Only highly skilled patching crews are able to obtain good feather-edge patches with the premixed material, and only the top course material can be used for this feather-edge patching.
5. During the late fall, winter and early spring seasons when the majority of the patching is necessary the premixed material is difficult to handle unless it can be obtained directly from the plant in small quantities as needed, which is impractical from the standpoint of the large plant producers.
6. It is never practical to use the premixed type of surfacing or patching material unless it can be taken from the truck and deposited directly where it is to serve its purpose, because of the costly rehandling and manipulation expense, particularly during cool weather. Furthermore, the material being bituminous coated cannot be placed in stock piles along the roads without there being a considerable loss in that part which comes into contact with the ground.

From the above, I hope it will not be construed that I am not in favor of the use of premixed material, because I am really very much in favor of this type of material, but do feel that its use, for the most part, should be confined to the leveling, surfacing, and resurfacing of larger areas, rather than for the patching of small holes and breaks.

5. Patching the Holes

There used to be the mistaken idea that the patching of holes could neither be done in inclement weather when the pavement was wet, nor in cold weather in the presence of frost. There was also a tendency on the part of maintenance men to allow holes to accumulate and then patch them all at one time. The idea was that the patching could be done more cheaply in this manner, because of the larger volume of work and materials; but lost sight of was the fact that "a stitch in time saves nine." Many of the holes which in the beginning were so small that a handful of material would have sufficed, developed into large cavities which required many times as much material to fill. Furthermore, the discomfort to the motorist was entirely disregarded.

This lack of constant maintenance resulted in the existence of numerous holes in the pavements all the time and particularly during the wet and cold seasons. It has been found that patches placed during wet and cold weather remain in place almost as well as those placed in dry, warm weather, and even if some of them fail, the added expense involved is justified in view of the service to the traveling public together with the fact that delayed repairs create additional repairs which would otherwise not exist. The maintenance man's slogan should be ''Keep all holes out of all pavements at all times.''

The size aggregate to be used in patching holes should be governed by the depth of the holes to be patched. If the holes are deep, first fill the hole to within about an inch of the surface with knapped or crushed stone (local stone if available), and lightly choke this base of the patch with fine aggregate, then penetrate it lightly with bituminous material. Then fill the last inch with a small size better wearing aggregate, either premixed or poured-in-place with the bituminous material selected. Next, seal the patch, regardless of whether it is a premixed or a poured-in-place patch, with a light application of bituminous material, and allow this seal to extend beyond the edges of the patch in order to seal out moisture and air and to liven the old surface around the patch. Finally cover the entire treated section with a cover coat of finely graded sand-size aggregate. In the case of shallow holes, the same procedure should be followed, excepting of course the base course using the larger size aggregate.

Do not ever use too fine an aggregate or an excess of bituminous material in filling holes, especially deep ones, because such a patch, when softened by warm weather, will shove or roll under heavy traffic; however, always use fine sand-size aggregate in the final sealing of the patch.

If possible, all patches should be compacted with not less than a five ton roller; however, adequate compaction can very often be obtained by hand tamping, by repeated passing of the truck wheels, or by allowing the traffic to do the compacting.

A straight edge should be a part of the equipment for all patching crews in order to insure smooth riding patches. Careful straight edging will eliminate, either high patches which usually result in two new depressions, one on either side of the new patch ,or low patches which cause a breakdown of the old surface on either side of the new patch, or else hold water to accelerate deterioration.

Careful drying of the aggregate before mixing with the bituminous material is very important. Many failures of patches or resurfacing may be attributed to improper control of the moisture in the aggregate at the time of mixing. There is now on the market reasonably priced portable drying equipment which will do much toward eliminating failures because of too much moisture in the aggregate used in patching.

The patching of one-half the road surface at one time should also be given consideration for the following reasons: (1) It gives uninterrupted travel on one-half of the road surface. (2) It keeps the motorist from complaining about the picked up material marring his car. (3) It prevents the material from being kicked out of place before it is properly sealed and rolled, thus saving both material and effort. (4) It permits a minimum of exposure both to men and equipment which is an important safety factor, not only to the workman but to the motorist. (5) It saves time because when patching is being done over the entire surface width the workmen have to stop every time a car comes along. (6) It is conducive to better patches, because the patches are completed and ready for traffic before the section of road is opened to traffic.

Always be careful to not over-bituminize a patch—it is better to have to replace a patch than to have a "fat" one, because this "fat" patch with its bad looks, warm weather bleeding and slipperiness will stay with you indefinitely, furthermore it is a useless waste of material. This warning against too much bitumen should also apply to all leveling courses which are used to bring out depressions or other irregularities. Make the leveling courses lean and thus avoid having an excess of bitumen to bleed through the new surface.

As "an ounce of prevention is worth a pound of cure" much of the patching of holes and breaks can be avoided by spot sealing. Many of the bituminous surfaces may, as a whole, appear to be in good condition, but a careful examination will disclose numerous small dry and checked or cracked sections. All such places should be given a light application of bituminous material and covered with fine sand-like aggregate. When the extent of these sections justifies it, a pressure distributor should be used to apply the bituminous material, and where the sections are small and intermittent the material may be applied from small heating kettles with hand pumps, this latter is termed hand spot-sealing and is very effective. The bituminous material to be used for this sealing should preferably be a light material.

BASE REPAIRS

Base repairs can be kept to a minimum if proper preventative measures are employed; therefore emphasis is placed on this important item of prevention.

Drainage is the most important factor to consider in the maintenance of the base of a bituminous surfaced road, because the water must be kept away from the base and the subgrade. Proper side ditching does much toward accomplishing this, but it is also necessary to provide some means of quickly carrying the water, which has become trapped in the base and on the subgrade, out through the shoulders to the side ditches. The rapid removal of this water is particularly important on new base and surfacing jobs, because there is much more chance for the accumulation of such water before the base, surface, and shoulders have become set.

Properly functioning shoulder drains have proven to be the most effective method of leading this trapped water out to the side ditches.

These shoulder drains should be at least 12″ wide and of variable depth. The bottom of the drain at the edge of the pavement should be from 2″ to 3″ below the top of the subgrade. The grade of these drains should be steep enough to draw the water from the subgrade. The most important location for such shoulder drains is at the low points in the grade line, at which points they should be placed perpendicular to the edge of the pavement. On grades the drains should be placed at an angle with the pavement edge in order to give effective drainage. The drains must be filled with loose stone or gravel in order to allow the water to get through and at the same time not interfere with the use of the shoulder.

These shoulder drains on the hills intercept a great deal of the surface edge water as well as the subgrade edge water, thus serving the dual purpose of draining the base and preventing shoulder erosion at the edge of the pavement.

It is not enough to merely install these shoulder drains—they must be maintained in such a condition that they will continue to function after rains and during the wet season.

In numerous instances it will be found necessary to extend these shoulder drains into the center of, or clear through, the pavement in order to drain wet-weather springs and the like. Drains of this type usually have to be deeper and wider than the regular shoulder drains.

The other important feature of base and subgrade drainage is a properly functioning system of drainage ditches through the medium of which the water table can be kept at an elevation lower than the top of the subgrade.

When a section of base does fail, the only solution I know is to remove the part which has broken up and replace it with new material. If the subgrade under the failed section is soft, which is usually the case, then this soft material should be removed and replaced with suitable material. It will be found necessary, in most instances, to replace this soft subgrade material with a subbase instead of new subgrade material. In such cases an outlet drain must be provided to draw off the subgrade water which is sure to accumulate; as a matter of fact, a good shoulder drain is always necessary where a base failure has been repaired.

The following method has been successfully used in stabilizing the surface of local creek gravel roads, preparatory to the placing of light or heavy road mixes:

First, the surface of the gravel should be bladed to a reasonably true section. Next, blade in from the sides two small windrows of earth about 6″ high and place one at each edge of the proposed surface. Then spread on this gravel subgrade, at the rate of approximately 80 lbs. per sq. yd. crushed aggregate, (limestone, hard sandstone, slag, etc.) Size No. 8, see table. Blade and drag this stabilizing course of aggregate until it works into the existing gravel surface. (Heavy traffic and wet weather materially expedite the working in process.) When the aggregate has all worked into the surface, apply ½ gal. per sq. yd. of prime tar (RT-3 or

RT-4) in two ¼ gal. applications, being careful to apply the second quarter gallon before the first application has become entirely dry. The surface, when thoroughly dry, is now ready for the application of the road mix, the weight of which depends upon the amount of money available; however, it is recommended that not less than a 55 lb. mix be used.

The criterion we use to determine when to patch a bituminous surface is immediately when a hole or break develops—regardless of weather conditions.

The criterion we use to determine when to seal a bituminous surface is when the surface becomes checked with fine cracks, and also shows signs of becoming dry with a light gray color from oxidation and evaporation.

The criterion we use to determine when to reconstruct a bituminous surface is when the surface has become distorted to such an extent that it is impossible with routine maintenance to maintain a reasonably smooth riding surface.

Maintenance of Bituminous Mats Where Edge Failures Have Been Due to Subgrade Failures

By

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In this discussion of maintenance of bituminous mats, I wish to refer to a condition which exists on all types of bituminous surfaces on South Dakota Highways ranging in thickness from 3″ mats to 1″ mats which have been in service for from 2 to 6 years. In line with this, I would say that prior to the last three years the preparation of base course prior to the construction of a bituminous mat was not given the consideration that it has been in the past few years with the result that failures on the older mats are quite prevalent.

Will say that this condition is pretty true throughout those sections of the State where the type of soil is such that it lends itself readily to capillary action. In wet weather it is of such a nature and it has a high shrinkage factor that the result is that during dry weather it opens up with cracks possibly 2″ wide and several feet deep. Due to the prevalence of this cracking when moisture does come it is fed quickly and readily to great depths which sets up a condition which causes unstable subgrade condition along the edge of the mats.

The matter of the edge failures being quite prevalent throughout the districts where this type of soil prevails can be traced to the fact that with a soil condition existing as has been stated that any moisture falling upon the mat during rain or snow removal that would place high banks along the slopes and would create an abnormal condition along the edges. It is a result of mat acting as a roof and dropping off all of this moisture over the edge and this edge being in such condition that it quickly absorbs this excess moisture and does not head it to the ditch for run-off. It has been thought to be advisable that if such a scheme could be worked out to a practical standpoint to waterproof these slopes in some manner so this soaking action could be stopped. There has been no feasible means developed for doing this. Any effort to protect this shoulder with tack coat oil has been defeated by the cracking action mentioned before.

Edge failures on bituminous mat over such a type develop first as a depression caused by heavy traffic pounding close to the edge of the mat for the reason that these mats are of only 20 foot width. Following this depression due to the fact that such depressions hold water, the mat soon becomes the alligator type through all these areas. With the seal broken and the mat opened up, water then is quickly fed through the base course and to the subgrade. With the subgrade softened from moisture being fed through the mat and also by the softening of the subgrade due to the conditions mentioned elsewhere in the discussion from capillary action and saturation from shoulder moisture, there is a complete breakdown of the mat. The extent of this is generally from 4 to 6 feet in width.

The old style of base course which was placed prior to mat construction of approximately 1,600 cubic yards per mile of crusher run gravel surfacing. There was no particular attention paid to the P.I. or binder and in many cases this gravel surfacing had been used as a gravel road for several years prior to the construction. With this being the case, the thickness of the base course was not at all times uniform. The existing graded sections over some of this type of work being old type also have shallow and narrow ditches.

The method of repair which has been practiced on this type of failure during the past season has been to break up and salvage the outer 6 foot edge of mat. This break up is made by first scarifying the width decided as necessary to salvage and then with power patrols carry this salvaged bituminous mat material to the opposite side of the road. The preparation of the subgrade is then taken through. With the oil condition being as described above, it is deemed advisable to change the texture of this soil to some extent by the addition of granular material. This action was gotten by a sheepsfoot roller being used to drive the existing base course material into the subgrade. In some cases it was found that the subgrade was of such a plasticity that without any difficulty the base course material was driven well into the subgrade and this compaction with sheepsfoot rollers was continued until the roller walked out of the material and the maximum compaction obtained by the use of this equipment. The resulting subgrade, of course, would then be considerably lower than the original base course due to the compaction action which had a tendency to shove the shoulder out.

Following this compaction which was made to the maximum density stabilized base course gravel was then placed on these edges. This gravel base course material was stabilized with clay and water and rolled to a density of approximately 135 lbs. per cubic foot. Following this operation the tack coat of bituminous material was placed upon this compacted edge existing of approximately ¼ gallon of SC-1. The repaired base course was now ready for the placement of the mat.

The method of preparing the mat for this repair edge was as follows: The salvaged mat material was pulverized by the use of discs and hard-faced rollers which caused a pulverized action on the existing mat on the road. As a result of this a very good result was obtained. When the salvaged material was well pulverized and was added to a sufficient quantity to take care of any shortage and for some additional width and thickened edge, this material was added to the roadway in a premixed condition. The premixing of material for such additions was accomplished in stock pile preparation at a central point.

The method mentioned above was used for the reason that it was found that the addition of material to the existing salvage windrow failed so throughout its length that it was practically impossible to mix the dry material and the existing salvaged windrow in a uniform method. The application of oil would be so irregular that it was found impossible to avoid having fat and lean spots. With the addition of material from stock piling it was found that a light application of oil to the compiled windrow would give a very good mix and very uniform.

Another reason which was used for the above mentioned method was that there was no inconvenience to traffic during the mixing operations and little oil applied on the project. The method of applying on one side of the road so that traffic could be handled through the project at all times. In order to further aid traffic in getting through the highway was worked on one side for a distance of one mile at a time. During all times of construction when windrow was on the road and edge being worked, traffic was handled through one way at a time with a pilot car which did not cause more than 10 minutes delay at one time. The experience was very good in line with traffic service and as there was no fresh oil encountered by the travelling public, there were no comments in the way of criticism of the method used.

When the compiled windrow was placed on the area on which the soil had been recently compacted by rolling with a steel faced roller in the same manner in which any bituminous mat would be laid. Following this operation in due time seal coat was applied.

The type of equipment used for this operation was that equipment necessary for any operation for handling bituminous mat construction under the road mix method. Material was supplied in stock pile for preparation of bituminous mixed material and also additional base material from regular plant. The only item which would be in addition to a regular bituminous mat construction outfit was use of the sheepsfoot roller which was the one piece of equipment that was quite important in obtaining the result that was desired for this repair.

The cost of the above work based on a 3″ mat replacement with 2 edges repaired to a width of approximately 3 feet to edge side was approximately $1,500 per mile.

It might be said here that there was an attempt to skin patch this same type of failure a year ago this season over these same areas with premixed material taken from a stationary plant. These patches were placed in all depressions which had shown failure as indicated in this discussion along the edges. In the placing of this skin patch type of material, the policy followed was to build up all depressions to a point where they did not hold water. After having these skin patches in place about one year it was found that practically in every place alligator type cracks developed and due to traffic shoving this material further into the subgrade.

The type of patching earlier in the discussion is apparently the answer for this type of failure. The only point left unsolved is the matter of waterproofing the slopes along the type of soil mentioned above which would stop capillary action immediately over the edge of the bituminous mat.

With reference to the type of bituminous maintenance which is rapidly coming to the foreground in our State is that in connection with those bituminous mats which are going out with the aeration of the bituminous materials with the result that the mats are not flexible enough to withstand traffic action and are breaking up into this action which is no apparent failure of the subgrade nor of the base course. The resulting cracks are not of the alligator type but are of the type which would seem to be a structural failure of the mat itself. The method of repair so far on these has been to crack seal these cracks by and wherever it is possible to do so and where the cracks are so uniform the method used has been to double seal with the distributor and sand the surface.

The process seal has been tried on one job of this type but we are not able as yet to tell the result of this type. A question that I would like to place at this time has reference to the advisability of remixing these mats and adding bituminous material to give life to these mats for extending service.

With respect to the criterion that we use to determine when to place seal or reconstruct a bituminous surface, will say that the policy of this State has been to start patching as soon as breakups appear on the surface. We have not let our bituminous mats continue in a broken up condition over a long period for the reason that it has been the policy here to not reconstruct but rather to maintain the surfaces as they exist. We have placed some retread type of approximately 1″ in thickness over some mats that have become badly broken up and have gotten fair service from these.

. With respect to the criterion for sealing, the policy has been that as soon as the bituminous material has worn from the aggregate so that the surface has shown a white appearance, it would indicate the absence of bituminous material over the surface, these jobs have been placed on the program for resealing. As practically all of the mats used in this State are of a temporary grade type, a seal coat is merely a wearing course and is not a water repellent necessity.

Highway Maintenance in California

By

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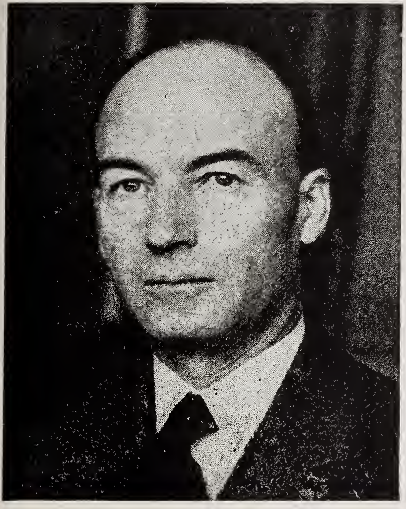

OUTLINE

- A. Shoulder Maintenance.
  - a. Penetration Method.
  - b. Roadmix Method.
  - c. Plantmix Method.
- B. Surface Repairs.
  - a. Patching.
    - Base Failures.
    - Surface Failures.
    - Premix vs. Penetration Patching.
      - Costs.
      - Equipment.
      - Serviceability.
      - Weather Resistance.
  - **b.** Surface Treatments.
    - Non-skid.
    - Seals.
      - Various Oils Used.
      - Various Covers.
      - Omitting Cover.
    - Blankets.
      - Armor Coats.
      - Retreads.
      - Roadmix and Plantmix Blankets.
      - Blanketing Old Portland Cement Concrete.
        - Rough and cracked—No movement of segments
        - Segments—curled and tilting.
  - c. Bituminous Retreatment.
    - Raveling, cracking and disintegration.
      - In initial stage — seal.
      - In intermediate stage — blanket.
      - In incipient failure stage — reprocess.
    - Corrugations, due to moisture.
- C. Base Repairs.
- D. Winter Maintenance.
- E. Plant Mix vs. Blade Mix.

A. SHOULDER MAINTENANCE

Shoulders in California are bituminous treated primarily for stabilization and the emergency use of traffic. Although base saturation is reduced by such treatment, this is not the chief purpose in oil treating shoulders.

Under California climatic conditions, only in a relatively few localities does vegetation encroach within the oiled shoulder limits.

In past years, the penetration method was used until an appreciable oil mat was obtained. At the present time bituminous shoulder work consists of reprocessing by oilmixing existing shoulders with an additional amount of liquid asphalt, building up settled though otherwise stable areas with plantmix material, or sealing existing oiled shoulders, otherwise stable, against the entrance of moisture through the surface.

In some locations where the natural material lacks stability, shoulders 2 to 6 feet wide, and 4 to 8 inches thick, are constructed of 1½ inch to dust crusher run or pit run gravel. The base is watered and rolling is done with power rollers where possible. If shoulders are too narrow to permit the use of a roller, compaction is obtained by truck rolling. A prime coat of about ¼ gallon SC-1A is applied to the finished base and followed by a seal of SC-6 applied at the rate of ¼ gallon per square yard and covered with 25 pounds of ⅜ - ¼″ screenings. In some cases, the grade of the base is left low to permit the placing of approximately two inches of plantmix surfacing. Although designated as shoulders, this latter type may be considered as widening, inasmuch as it is nearly equal in bearing power to the main traveled way.

a. Penetration Method

The initial treatment of earth shoulders is usually by the penetration method and may be considered as the first step in "stage construction." The choice of liquid asphalt and the amount of the application depends on the type of soil encountered. On fine or tightly bound soils, SC-1A is applied at the rate of ¼ to ½ gallon per square yard and covered with suitable roadside material, if available; otherwise a select material or sand is imported for the purpose. Where possible, the liquid asphalt should be allowed to penetrate, covering only those portions where free oil remains. The cost of this type of work averages between 3 and 5 cents per gallon in place.

On the more porous or coarsely graded materials, the heavier SC-2 should be substituted for the SC-1A at approximately the same rate of application. Where the oil cake is dry but intact, a seal of SC-3 or SC-4 is applied at the rate of 1/6 to ¼ gallon per square yard and covered with either roadside or imported material. In desert areas, the "life" of the mat has been successfully renewed by applying SC-2 at the rate of 1/12 to 1/6 gallon per square yard without cover.

b. Roadmix Method.

The next stage in the development of the oil treated shoulder is the roadmixed type which is applied when:

1. The oil mat as furnished by the penetration type is insufficient to withstand traffic.
2. The oil mat through repeated penetration applications has attained sufficient thickness but has become distorted.

On the other hand, roadmixing has been the initial method in certain locations where the shoulders are subject to extraordinary use by traffic and it was evident that the penetration method would prove inadequate.

On existing untreated or treated shoulders, the material is scarified to the required depth. If moisture is present, the loosened material is allowed to dry out before applying the oil. The quantity of liquid asphalt depends on the type of material and depth of treatment, but will ordinarily not be less than ½ gallon per square yard.

In the initial treatment, the grade and percentage of liquid asphalt should be determined from laboratory tests. Climatic conditions, as well as the material used, influence the selection of type of oil. Slow curing asphalts, either SC-2, SC-3, or SC-4 are generally used, but MC and ROMC types give good results where the natural soil is of good quality. On fine loams or silts, the SC-2 has given best results for the initial treatment. With coarser materials or where severe winter conditions prevail, the heavier oils such as SC-4, ROMC, etc., are recommended. If the base is loose or porous, it should be primed with SC-1A or SC-2. Roadmixing is done with tow graders, traveling mixers, motor graders or multiple-blade sled-type mixers. Only pneumatic tired motor graders, without lugs, should be used for spreading and finishing. Compaction may be performed by traffic, 8 ton rollers or pneumatic tired multiwheel rollers, depending on depth of mix.

Where existing material is unsuitable, select material may be imported and roadmixed as above.

In maintaining roadmixed shoulders which have become distorted without base failure, a disc scarifier is used to loosen and break up material to a restricted depth, which is approximately one half the thickness of the cake. This eliminates the difficulty experienced in scarfying full depth and disturbing and incorporating the base material in the mix. Further advantage is gained by reducing the amount of oil required for remixing, thus confining the richer mix to the top where it will be most effective in reducing ravel and preventing water saturation from the surface.

To the material thus scarified and pulverized, SC-2, SC-3 or SC-4 is applied at the rate of 1/5 to ¾ gallon per square yard. Mixing is done with a multiple blade shoulder mixing drag (as per sketch attached) or with motor graders, depending on the equipment available. The multiple blade mixing drag will mix the material in about 25 per cent of the number of passes required by a motor grader. This special treatment costs approximately two to four cents per square yard.

Costs for roadmixing vary greatly. For machine mixing, the cost, including oil, is approximately three to five cents per square yard, one inch thick.

Roadmixing with graders costs between six and ten cents per square yard, one inch thick. The multiple-blade mixing costs less than that performed by motor grader and can also be used on shoulders of restricted width with less interference with traffic.

c. Plantmix Method

Plantmix has been used in heavy traffic areas and for leveling sunken shoulders which have not actually failed. SC-4, ROMC and MC liquid asphalts may be used as binders. Plantmix can be placed for $4.50 to $6.50 per ton.

Various gradings used for plantmix appear in the following table:

TABLE 1

GRADING REQUIREMENTS FOR PLANT MIX SURFACING

Sieve Size	Per Cent Passing for Various Gradings			
(Square opening)	¾″ - Dust	½″ - Dust	⅜″ - Dust	No. 4 - Dust
1 inch	100			
¾ inch	95–100	100		
½ inch		95–100	100	
⅜ inch	65–85	80–95	95–100	100
No. 4	50–65	58–73	65–85	90–100
No. 16	30–43	33–45	38–52	46–67
No. 50	18–27	18–27	22–30	25–36
No. 200	6–10	6–12	8–15	10–17
Approximate amount of liquid asphalt*:				
Lower Limit	3.9	3.9	4.2	4.5
Upper Limit	4.6	4.7	5.0	5.3

Moisture content in the plant mix should not exceed 1%. Should it be in excess of this amount, the plantmix material should be bladed back and forth until it has been reduced to the above figure.

Filler material should not exceed 25% separation, as determined by the Water-Asphalt Preferential Test. Likewise, the stability of the mix should not be less than 35% when tested with the Hveem Stabilometer. Swell should not exceed 0.030″ as determined by the Swell Test (immersing oil mix briquette in water).

The aggregate may be:

1. Broken stone or crushed gravel.
2. Natural material essentially the same in surface quality and angularity, or
3. Natural rough surface meeting the requirements for stability.
   It should be tough, durable and sound, and free from adobe, vegetable matter and other deleterious substances.

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*The amount of liquid asphalt will vary from quantity shown for porous rock or different fines. The amounts shown are satisfactory for commercial plants. For ordinary conditions, SC-3, SC-4, MC-3 or MC-4 grades of liquid asphalt should be used.

The hardness of the aggregate shall be:

1. By Wet Shot Rattler Test—loss not more than 55%.
2. By Los Angeles Rattler Test—loss after 500 revolutions not more than 45%.

Where thickness of plant mix is less than two inches, a prime coat of ¼ to ½ gallon of SC-1A or SC-2 shall be applied and allowed to thoroughly penetrate the prepared base.

For leveling courses on existing sunken oiled shoulders, a tack or paint coat of emulsion or RC liquid asphalt should first be applied at the rate of 1/10 to 1/12 gallon per square yard. The plant mix should be spread with satisfactory spreader boxes adjustable for width and depth, or by truck tail gate, and finished with motor grader. When fully compacted, the grade of the shoulder should be flush with the adjacent pavement edge.

B. SURFACE REPAIRS

a. Patching

1. Base Failures:

Where the failure is confined to the pavement edge due to lack of sufficient base, it is our practice to remove the surface and base, salvaging the latter if not impregnated with clay.

The sub-base is then excavated to secure the additional thickness of base required, which may vary from six to eight inches, depending upon the subgrade support. Local materials, if available and satisfactory, are used in providing this added thickness. The material is placed and compacted in not to exceed three inch layers, being watered and compacted with a power roller. The salvaged base is then added and compacted in a similar manner. The surfacing which has been broken up is retempered with an SC, ROMC or MC cutback, remixed and laid.

On smaller areas having failures of a nature similar to the above where the use of mobile equipment is prohibitive, the same is done by hand, including the compaction of both base and surface, with a hand tamper. However, truck wheels may be used in compacting the top course. On small areas requiring hand work, new premix or penetration material is used to replace the old.

2. Surface Failures:

Small ravelled areas are primed with emulsion, SC-6 or RC liquid asphalts, after loose material has been removed and the hole shaped with sloped sides. Patching material may be either plant mix, road mix or clean screenings, penetrated with SC-6 or emulsion binder. The liquid asphalt in the premixed material should be similar to that used on the surface to be patched. On the lower type oil surfaces, roadside or other suitable local materials which have been premixed by the road mix method are used for patching, with satisfactory results. Tamping material in patched area may be done by hand tamping, loaded truck wheels or portable hand roller. Surface of the completed patch should be flush with the adjoining pavement.

For penetration patching, courses are built up in the same manner as for a bituminous macadam, placing coarser rock in bottom and keying such course with a finer grading of rock. A minimum of asphaltic binder should be used in the bottom courses.

For the final course, the top is screened. Rock should be clean and dry when used. For shallow depressions, use aggregate sized to slightly less than the depth of the hole.

Generally, the type of patching material should be similar to the existing pavement. However, both penetration and premix types have been used successfully in patching and leveling asphaltic and Portland cement concrete pavements. Equally good results have been obtained in tapering edges of leveling patches with the use of both types of patching.

On the more rigid type surfaces, where cracks have developed in the pavement and along the edge adjoining the bituminous treated shoulders, a bituminous filler is used to seal the cracks and prevent the entrance of water into the base. This filler is usually a blend of 25 to 30% SC-6 liquid asphalt and 70 to 75% 31-40 penetration air blown asphalt.

3. Premix vs. Penetration Patching:

In California, two types of premix are used. In the outlying areas where satisfactory aggregate is accessible the material is roadmixed with the desired type of liquid asphalt and stockpiled for future use in patching. Where no satisfactory local material is available and in areas where commercial plants are located, the plantmixed material conforming to specification requirements is purchased and stockpiled at convenient locations.

Where satisfactory local materials are available, the roadmix type of patching is less costly than either the plantmix or penetrating type. Our price for the materials roadmixed and stockpiled varies from $1.00 to $2.00 per ton, which is cheaper than we are able to obtain plantmix at any location.

Serviceability of the premix and penetration types of patching is about equal under average conditions, but in prevailing frost areas the penetration type of more than one rock in thickness has been less successful, due to ravel and disintegration.

A low cost method of patching is practiced on roads having a light oil mix (SC-2 or 3) surface, of about one to two inches in thickness.

One man equipped with a small orchard type hand spray outfit of about 3 gallons capacity, applies a light tack coat application of emulsion after the holes to be patched have been cleaned of all loose material. A truck loaded with premix follows, and one or two men in the truck cast the premix upon the tack coat. Another laborer follows up with a rake, leveling and smoothing the material to be compacted by traffic.

Comparison of average costs of different types of bituminous patching is shown in the following table:

Averaging Patching Costs

Per Square Yard, 1 Inch Thick

Oil Type	Premix		Penetration
	Roadmix	Plantmix	
SC-2, SC-4, ROMC	.12 to .25	.20 to .36	
SC-6			.22 to .35
Emulsified Asphalt			.20 to .25

	Premix		Penetration	
	Roadmix	Plantmix	SC-6	Emulsion
Foreman	1	1	1	1
Truck Drivers	1–2	1–2	1–2	1–2
Laborers	3–6	3–6	3–6	3–6
2-ton Dump Trucks	1–2	1–2	1–2	1–2
190-300 Gals. Emulsified asphalt pressure kettles	1	1		1
Hand Roller	1	1	1	1
Hot Oil Kettle			1	

Although the same organization may be used in all types of patching, greater progress is usually obtained in placing the premix. The size of the crew and the number of equipment units depend on the quantity of work to be done in a day.

b. Surface Treatments

Surface treatments are confined to those operations which leave the existing surface undisturbed and which are intended to improve riding quality, increase bearing power, seal for abrasive resistance or moisture exclusion, or provide a non-skid surface texture.

1. Non-Skid:

Non-skid treatment is confined to areas which have become slippery and dangerous to traffic through the flushing of excess asphalt to the surface under the action of weather and traffic. In most cases, it is necessary to remove the excess oil by discing or planing with heater or chisel tooth planer before application of the non-skid top. This preliminary treatment also provides an improved riding surface by removing irregularities and corrugations.

The non-skid texture may be effected by the placing of a plantmix blanket, an armor coat, retread or crushed rock with asphaltic binder consisting of SC-6 or emulsified asphalt. Excellent non-skid quality has also been obtained with natural rock asphalt placed in a layer not over 1½ inches thick.

The most generally used non-skid treatment in California consists of an application of 1/6 to 1/7 gallon of SC-6 penetration type emulsified asphalt, covered with 30 to 35 pounds per square yard of clean and extra hard screenings, ½″ to ⅜″ in size. The surface should first be cleaned thoroughly by brooming. A better distribution of emulsion over the old surface may be obtained by first applying water at the rate of about 1/15 gallon per square yard.

Bituminous binder should not be spread to a greater distance in advance of the screening operations than can be covered with screenings before the emulsion breaks.

Screenings should be placed with an approved spreader box capable of applying the material at a uniform rate over the full width of the binder.

Immediately after applying screenings, the surface should be rolled with a 6-ton power roller. Necessary hand spotting and brooming required to obtain an even distribution of material should follow and the surface again rolled. Traffic should not be permitted to use the newly treated surface until the binder has thoroughly set. After it is opened to traffic, any excess screenings should be removed with a revolving broom to prevent corrugations and reduce hazard from flying rocks under the action of traffic.

2. Seals:

Seals are similar to the non-skid treatment, except smaller screenings are used for cover, the prime purpose being to prevent entrance of water into the base and eliminate surface ravel.

Base failures and surface depressions should be corrected well in advance of the sealing to allow complete curing and compaction of the patches. Rich spots in the surface should be removed by heating with pavement heater. If necessary to improve the riding surface, discs, chisel tooth or heater planers should be used to remove the high spots of a distorted surface.

Immediately before placing seal, surface should be cleaned with power and hand brooms.

The grade and amount of liquid asphalt and the size of the screenings should be such that maximum temperatures and heavy traffic will not cause excessive bleeding.

For example, it has been our experience that the softer asphalts such as ROMC, SC, etc., may be used in the Pacific Coastal areas without bleeding. Where high summer and mild winter temperatures obtain, the harder MC and RC liquid asphalts have proved satisfactory. Where the winter temperatures are severe, the more ductile ROMC and SC-3, 4 and 6 should be used. The SC-2 oils are not entirely satisfactory in winter areas, and the SC-6 bleeds under high summer temperatures. Emulsion seals if properly applied have given good service, regardless of weather and temperature conditions.

Typical seals placed in California:

Class A. Type of liquid asphalt:
SC-2, 3, 4, RC, MC or ROMC.

Amount of application, liquid asphalt:
.15 to .25 gallon per square yard.

Cover application:

10 to 15 lbs. sand, or 15 to 25 lbs. ⅜″ - ¼″ or ¼″ - ⅛″ screenings.

Class B. Type of liquid asphalt:

SC-6.

Amount of application:

0.17 gallon per square yard.

Cover application:

25 lbs. ⅜″ - ¼″ screenings.

This type is also placed as a double seal to improve riding qualities on pavements having slight irregularities, and to extend the life of the non-skid properties.

Double Class B. Type of liquid asphalt:

SC-6.

Amount of application:

First: 0.15 gallon per square yard.

Second: 0.13 gallon per square yard.

Cover application:

First: 23 to 28 lbs. ⅜″ - ¼″ screenings

Second: 12 to 16 lbs. ¼″ - ⅛″ screenings.

Class C. Type of liquid asphalt:

Penetration type emulsified asphalt (SC-6 base).

Amount of application:

0.17 to .20 gallon per square yard.

Cover application:

12 to 20 lbs. ⅜″ - ¼″ or ¼″ - ⅛″ screenings, or 10 to 15 lbs. sand per square yard.

Cover material should be placed before the setting of the liquid asphalt or emulsion occurs. It should then be hand spotted, rolled with a 5-ton roller, broomed and again rolled. The initial rolling should follow immediately after spreading the cover material. Brooming is continued until the seal has set up.

Excess cover material should then be removed before the formation of corrugations or displacement of imbedded cover under the action of traffic. Any showing of excess binder should be covered by additional screenings or sand.

The following table shows grading requirements for screenings and sand used in seal, non-skid, armor and retread treatments, as well as for patching. Too much emphasis cannot be placed on the use of clean washed material. Dust or silt in the screenings will reduce the adhesive qualities of the binder. Likewise, seals placed in cold weather become chilled, with a reduction in adhesion to both the pavement and the cover material.

TABLE 2

GRADING REQUIREMENTS FOR CLEAN CRUSHED ROCK AND SCREENINGS

Percentage Passing Sieves— Minimum and Maximum Limits
Designation, Nominal Size and Use

Sieve Size (Square openings)					Screenings Coarse	Screenings Medium	Screenings Fine	
	1½" - ¾" for heavy retread	1" - ¾" for retread, patching	1" - ½" for retread, armor, patching	¾" - ½" for retread, armor, patching	½" - ⅜" for armor, non-skid, patching	⅜" - ¼" for armor, seal coat patching	¼" - ⅛" for seal coat	Sand for seal coat
2"	100							
1½"	90–100	100	100					
1"		90–100	90–100	100				
¾"	0–10	0–10		90–100	100			
½"			0–10	0–15	90–100	100		
⅜"	0–3	0–3			0–15	90–100	100	100
No. 4			0–3	0–3	0–4	0–15	90–100	90–100
No. 8						0–3	0–15	0–35
No. 16							0–3	
No. 30								0–20
No. 200	0–1	0–1	0–1	0–1	0–1	0–1	0–1	0–3

All material except sand is to be washed.

Modified seals under maintenance and the correction by skin patches of small areas of pervious, checked, or slightly raveled surfacing may be used, using one of the following methods:

TABLE 3

Method	Type of Asphaltic Material	Rate of Application/Sq. Yd.	Cover Material Sq. Yd.
1.	SC-2, SC-3, SC-4 or SC-6 emulsified asphalt	1/10 to 1/5 gal.	5 to 15 lbs. roadside material or sand.
2.	90–95 (SC-6) or SC-6 emulsified asphalt	1/6 to 1/5 gal.	17 to 25 lbs. medium screenings.
3.	SC-6 emulsified asphalt, penetration type.	1/7 to 1/5 gal.	5 to 15 lbs. fine screenings or sand.
4.	Cutback asphalts of MC or ROMC type	1/6 to 1/5 gal.	5 to 15 lbs. fine screenings or sand.

Seals without cover: This practice is observed more as a construction than maintenance method, because of the inconvenience to traffic. The method has been used successfully on shoulders where traffic can be protected. Liquid asphalts have consisted of SC-2 or SC-4, MC-2 or RC-1, applied at the rate of 1/16 to 1/5 gallon per square yard.

3. Blankets:

Armor Coats: An armor coat provides a thick, non-skid and tenacious wearing surface, and also improves the riding qualities of an existing pavement. It consists of clean screenings graded in the manner of macadam rock, and placed with an SC-6 or emulsified binder in two or more courses.

Typical construction is as follows: The original surface is first swept with a power broom and treated with 1/8 gallon penetration type emulsified asphalt, then covered with 60 pounds ½″ - ⅜″ screenings, and rolled with a 5 to 8 ton roller after spreading to a smooth even surface with a blade or long base drag. A second application of emulsion at the rate of ⅜ gallon per square yard is followed with 25 pounds of ⅜″ - ¼″ screenings. The surface is again rolled and broomed until the rock particles are set. The final application consists of ¼ to ⅜ gallon emulsion covered with 15 pounds of ¼″ - ⅛″ screenings, which is also broomed and rolled until it is thoroughly compacted.

If a lighter type armor coat is desired, it may be placed in two courses 25 pounds ½″ - ⅜″ screenings penetrated with ⅜ gallon emulsion per square yard and covered with 15 pounds ⅜″ - ¼″ screenings and leveled, broomed and rolled as for the heavier type.

Retreads: Retreads are used to provide a non-skid blanket with improved riding qualities, and to furnish a tenacious flexible surface over a somewhat shattered rigid type pavement. This type is superior to the armor coats listed above.

Retreads differ from armor coats. The base course is heavier and is mixed with mixing type emulsion by means of motor graders or multiple blade retread mixers especially designed for the purpose. The intermediate and top courses are placed by the penetration method in a manner similar to the armor coats. Gradings and quality of screenings are the same as those listed in Table 2 under the topic ''Seals.''

Retreads are constructed in four different thicknesses as outlined in Table 4, on the following page.

Roadmix and Plantmix Blankets: These types of blankets are usually used to level and reinforce existing surfaces which have become distorted or proved inadequate under traffic.

The choice of bituminous binder depends on climatic conditions, the characteristics of the aggregate, and type and condition of the underlying surface. It has been found that the SC liquid asphalts produce a more flexible blanket and reduce the transmission of the cracks from the original pavement.

The material requirements are the same as for the mixes used and described previously under ''Shoulder Maintenance, Roadmix and Plantmix Methods.'' In general, mixing operations are similar, although in some locations travelling mixers and mechanical finishers have been used successfully.

Where the material is roadmixed on the pavement or roadside, great care should be exercised to prevent the deposit of film dust or free oil between the completed blanket and the original pavement. The presence of dust has been responsible for slipping and shoving, and the free oil has saturated the blanket, resulting in rolling and bleeding.

It has been found advisable in locations where the annual precipitation exceeds 10 inches, to place a seal on this type of blanket, particularly where the subgrade is of adverse material.

Blanketing Old Portland Cement Concrete Pavements, Rough and Cracked—no movement of segments: As previously mentioned, the use of roadmix and plantmix blankets have been found satisfactory if the less viscous SC liquid asphalts are used as binders. Likewise, good results have been obtained with the use of armors and emulsion retreads because of their greater flexibility.

Segments—curled and tilting: Bituminous blankets placed directly upon pavements in this condition have invariably developed cracks transmitted from the original pavement. Disintegration follows and requires excessive patching.

Better and more permanent results are obtained by placing a cushion course of untreated gravel or crusher surfacing, after which either a prime and seal coat or a plant or roadmix may be placed as a wearing surface. Vehicle loads are distributed over a greater area, and any movement of the original pavement is absorbed. The thickness of the cushion course depends on traffic loadings, condition of pavement and subgrade. Usually the minimum thickness is 4″ to 6″, and is increased to 12″ or more where the presence of alkali or ground water is responsible for base failures.

TABLE 4

RETREAD TYPE SURFACING

Quantities per Square Yard

Procedure	Light Retread ¾ to ⅞″	Class A	Class B	Heavy Retread 2¼″	Remarks
1. Place clean, crushed rock, Apply mix type emulsion. Mix and level. Roll until compacted.	60 lbs., ¾ - ½″; ½ gal.	100–115 lbs. 1 - ¾″, 0.5 to 0.7 gals.	110–125 lbs. 1 - ¾″, 0.6 to 0.7 gals.	170 lbs. 1½ - ¾″; 1 gallon	Do not overmix. Spread before stripping occurs.
2. Cover with clean screenings.	25 lbs., ⅜ - ¼″	10–20 lbs. ⅜ - ¼″	15 to 20 lbs. ⅜ - ¼″	30 lbs. ½ - ⅜″	Apply evenly, roll, broom lightly, roll.
3. Apply penetration type emulsion. Cover with clean screenings.	¼ gal.; 15 lbs. ¼ - ⅛″	0.2 gal. 20 lbs. ⅜ - ¼″	0.2 gal. 20 lbs. ⅜ - ¼″	½ gal.; 25 lbs. ⅜ - ¼″	Apply evenly, roll, broom lightly, roll.
4. Apply penetration type emulsion, cover, broom, roll.	1/7 to 1/6 gal. 5–10 lbs. sand (no rolling)	0.25 gal. 12 lbs. ¼ - ⅛″	0.25 gal. 12 lbs. ¼ - ⅛″	¼ gal.; 15 lbs. ¼ - ⅛″	Apply evenly, roll, broom lightly, roll.
5. Apply penetration type emulsion. Cover, broom.		0.2–0.3 gal. 8–12 lb. ¼ - ⅛″	0.2–0.3 gal. 8–12 lb. ¼ - ⅛″	1/7 to 1/6 gal. 5–10 lb. sand	Apply evenly, roll, broom lightly, roll.
Approximate thickness	¾–⅞″	1¼″	1½″	2¼″	

The grading requirements for the crusher run material are as follows:

TABLE 5

Sieve Size Square Openings	Percentage 2″ to Dust	Passing ½″ to Dust
2½″	100	
2″	90–100	100
1½″		90–100
¾″	50–80	50–90
No. 4	30–50	30–50
No. 200	5–12	5–12

The material is placed in 3″ to 4″ layers, each of which is compacted with water and 10 ton power rollers or two-axle type pneumatic tired rollers. Brooming, watering and rolling are carried on simultaneously to insure a uniform thickness of grout on the surface. After surface is dried out, the fine material is swept with a power broom until the surface rock is exposed. The prime, consisting of ½ gallon of SC-1A per square yard, is then applied and allowed to thoroughly penetrate. The chosen wearing surface, penetration or mix type is then placed as already described herein.

c. Bituminous Retreatments
Raveling, Cracking and Disintegration

1. In Initial Stage:

Where raveling and cracking have just started, they can usually be retarded by the application of a seal. The choice of seal and the details of application have been discussed herein under (B) Surface Repairs,

(b) Surface Treatments, Seals.

2. In the Intermediate Stage:

Where raveling and cracking are more extensive, due to the abrasive action of traffic on an otherwise stable surface, a leveling course or blanket should be placed after priming the surface with emulsion or RC asphalt. This may consist of a light road mix or plant mix, or a suitable wearing course may be acquired through the use of armors or retreads.

In some cases, under light traffic conditions and where the surface contains one of the lighter liquid asphalts, further surface disintegration may be corrected by discing the surface to about one-half its depth and applying approximately ⅜ gallon of liquid asphalt per square yard, thoroughly mixing this with the loosened material, relaying and compacting under traffic. This method is inexpensive, reduces the amount of oil in reprocessing and confines the richer mix to the top.

3. In Incipient Failure Stage:

When the surface has failed to the extent that patching is not economical, it should be scarified to the full depth and pulverized. Areas which indicate base failures due to subsurface springs should be corrected by removing unsuitable material to a depth which will permit draining with perforated pipe or rock drains. Portions which have inadequate base thickness, should be reinforced with untreated gravel, crusher run or select local materials, after the scarified surface has been bladed to one side in a windrow. Priming of base is necessary if mat is less than two inches, or if raveling occurs under the action of traffic or construction equipment.

If moisture is present in the bituminous surfacing, it should be dried out by blading or other means before adding the oil. Liquid asphalt is then applied at a rate depending upon the type of material and depth being treated, but ordinarily in the amount of ½ to 1 gallon per square yard. Mixing may be done with blade graders or traveling mixers. Spreading and finishing should be done with smooth pneumatic-tired motor graders. Compaction should be procured in 1″ layers either by traffic, 8 ton rollers, or multi-wheel pneumatic-tired rollers. After thorough compaction and curing, the surface should be sealed, providing it lies within an area where the annual precipitation exceeds 10 inches.

On surfaces where corrugations and shoving occurred due to the presence of moisture in the mat, the treatment is very much the same as described in the preceding paragraph. In some locations, moisture corrugations have been corrected by light discing, which removes corrugations and permits the escape of a certain amount of entrained moisture. In some cases, this treatment has permanently corrected the condition. The same treatment has given temporary relief during cold weather when it is inadvisable to reprocess. In the areas of greater rainfall, however, disced surfaces should be resealed before the wet season.

C. BASE REPAIRS

The advisability of salvaging and reprocessing the existing oil mat during base reinforcing operations should depend on:

1. Value of the existing mat.
2. The desirability of retaining the mat as a membrane to cut off the entrance of ground water from below.
3. The retention of the compacted base which might be seriously disturbed by removal of the mat.

In cases where salvaging is justified, the mat is scarified full depth and windrowed clear of base operations. If the base is found to contain more than the optimum moisture, it should be exposed and dried out before additional base material is placed. If the base appears to be unstable at any locations due to ground water, pipe or rock drains shall be placed at such depth as to adequately drain the areas.

Shoulders should be restored and built up to conform to the grade of the new base to act as a choker for the placing of the base.

Material consisting of gravel or crusher run should then be placed to the desired thickness upon the original base, and watered and compacted in layers as hereinbefore described.

After the surface has dried sufficiently, it should be primed with ½ gallon of SC-1A or SC-2. The windrowed mat material should then be fluxed with a sufficient quantity of liquid asphalt to provide a workable mix having sufficient stability. The material should be thoroughly mixed, either with blade graders or mixing machines of the power or multiple blade types, and laid and finished with a smooth pneumatic-tired motor grader or mechanical finisher. Compaction may be made by traffic, power rollers or multi-wheel pneumatic-tired rollers. Shoulders shall then be restored to conform to grade of new surface.

Stabilization of Bases: California has made use of fine aggregate mixing emulsion in stabilizing existing bases and imported base materials. Ten projects, totalling over 25 miles in length, have been completed with varying success.

When warranted, the oil mat has been salvaged for replacement after the base operations were completed. In most instances, however, the surface was incorporated with the base and the resulting mixture stabilized.

In general, subgrade material in California requiring stabilization is of such fine texture as to require a high percentage of emulsion stabilizer, thus rendering such treatment prohibitive. As an alternative, either coarse aggregate or other select material is imported and incorporated with the portion of subgrade to be treated so as to acquire the desired grading, or imported material is roadmixed or plantmixed with emulsion stabilizer and placed upon the prepared subgrade. Existing gravel bases which have failed under traffic and adverse submoisture conditions, have been processed with the stabilizer and relaid and compacted, after additional binder material has been added, if required, to obtain the desired grading.

Typical stabilization projects are briefly outlined below:

Case 1. Road over delta land consisting of adobe overlying a peat formation. Highly cultivated area under irrigation with the ground water very near the surface during most of the year. Road had received only light applications of gravel at scattered critical areas. The wearing course was a light SC penetration type oil mat subject to annual disintegration and replacement.

Crusher run, 4 inches thick when compacted, was stabilized by the plant mix method, watered and rolled. After compaction, it was primed with SC-1A and sealed with SC-6 and fine screenings. During the first year, heavy trucking and winter conditions caused many failures, and it was found necessary to reinforce further with an additional 3″ of untreated crusher run base before the following winter. Both oil mats were retained as membranes to cut off the rise of water from the subgrade. The base was again primed with SC-1A and followed with a seal application of SC-6 and screenings. After three winters, the surface is in good condition.

Case 2. Road crosses bottom land which serves as a bypass for Sacramento River flood waters and for a period of several weeks may be covered by water to a depth of five to eight feet. An irrigation ditch, parallel to the project and immediately adjacent, carries water, the surface of which is only two to three feet below the top of the pavement during most of the summer and fall. Obviously the base and adobe subgrade is in a perpetual state of saturation.

When this section was a county road, several lifts of gravel had been placed and lost in the adobe subgrade. After its inclusion into the State system in 1933, 6 to 8 inches of imported gravel was placed and topped with a 4 inch road mix surface. Moisture and traffic brought general failures, and in 1937 permission was obtained from the Reclamation District to raise the grade of the road a limited amount. Imported gravel was road mixed with stabilizing emulsion, and placed in two compacted layers totalling 5 inches in thickness over the existing oil mix surface. The stabilized material was extended 6 inches thick and 4 feet wide each side of the 22 foot traveled way, in order to provide greater resistance to the current of the overtopping flood waters. The wearing surface consisted of a 22′ x 3″ ROMC road mixed gravel surfacing.

In 1938, following a severe winter, about one half of the project was reinforced by the addition of a 22′ x 2″ road mixed gravel blanket and the remaining portion was reprocessed and sealed, the stabilized borders on both sides being surfaced with road mixed gravel 2 inches thick and 4 feet wide.

This section of road is in much better condition at the present time, with only a few isolated failures having developed.

Case 3. This project consisted of stabilizing the 4 inch gravel base under a 4 inch plant mix surface. The subgrade material was peat, which became saturated with a rise of the ground water level.

The plant mix surface was scarified and bladed to one side. The base was scarified to a 4 inch depth, pulverized, and then mixed with stabilizing emulsion at the rate of 3% by weight of treated material or 15% by weight of portion passing the 200 mesh sieve. The emulsion was diluted with equal parts of water. After thoroughly mixing with tow graders, the material was spread to the original width and compacted by watering and rolling. The base was primed with stabilizing emulsion diluted with water in the proportion of 1 to 4.

The windrowed plant mix surfacing was then remixed with SC-4 liquid asphalt at the rate of ⅔ gallon per square yard, and relaid upon the stabilized base and rolled. Sealing consisted of an application of ¼ gallon per square yard of penetration type emulsion, covered with 28 pounds of ½″ - ¼″ crushed rock screenings.

Some failures have occurred, requiring reworking, but, in general, this treatment has proved to be satisfactory.

In nearly every instance, failures of this type of treatment may be attributed to excessive moisture remaining in the treated base, or to the lack of adequate thickness necessary for the support of superimposed loads.

The choice of any method of stabilization should be based on a thorough economic study. Where base thickness is of primary importance and suitable binder and crusher run or gravel base is available, the use of emulsion for stabilization would undoubtedly be uneconomical.

D. WINTER MAINTENANCE

Under severe winter conditions, holes and base failures are patched with untreated gravel or local material until warm weather, when permanent patches by the premix or penetration method are placed. In some localities where such local material will not compact, temporary patches are made with premixed material.

Premix is used almost exclusively where wet weather prevails during most of the winter. In areas of intermittent wet weather, emulsion and screenings may be used even if pavement is damp.

Premix material is stockpiled at convenient locations along the road. Road mixing is done during the summer oiling season. Plant mix material is purchased as required from commercial sources when available.

E. PLANT MIX VS. BLADE MIX

As discussed in Section B (a), 3, Premix vs. Penetration Patching, where local materials are accessible, the cost of road mix material is far below that of plant mix.

Advantages of the plant mix, however, are:

1. Mixing is more thorough, producing a more weather resistant mixture.
2. Mixing operations are located away from road with less hazard and inconvenience to traffic.
3. Heavier oils may be used during cold weather.
4. Proportioning of oil and aggregate can be more closely controlled.

Advantages of the blade or road mix are:

1. Better in isolated areas where this type of equipment can be used economically or is easily accessible.
2. When using maintenance forces for production, the operators are more familiar with this type of equipment, which is also used in other maintenance operations.

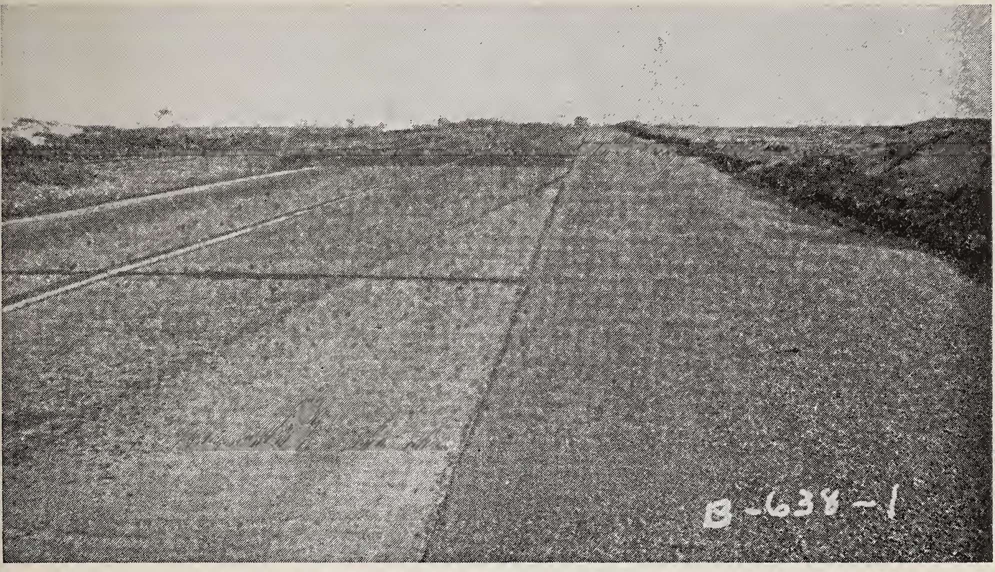
Summarizing 25 projects, the cost of constructing road and plant mix surfaces 20 feet wide and 1 inch thick, is given in the table below. The annual maintenance cost per mile is also included:

TABLE 6

Type	No. of Locations	Total Length Miles	Average Cost Construction 20′ x 1″	Per Mile Maintenance	Age Years
SC-2, Road mix	5	41.55	$ 695	$238	4-11
MC, ROMC, Roadmix	4	12.76	1,308	394	4-6
SC-2, Plant mix	4	42.5	1,614	215	6-9
MC, ROMC, Plant mix	8	65.18	1,711	270	4-6
Emulsion Retread	4	22.70	2,690	86	5

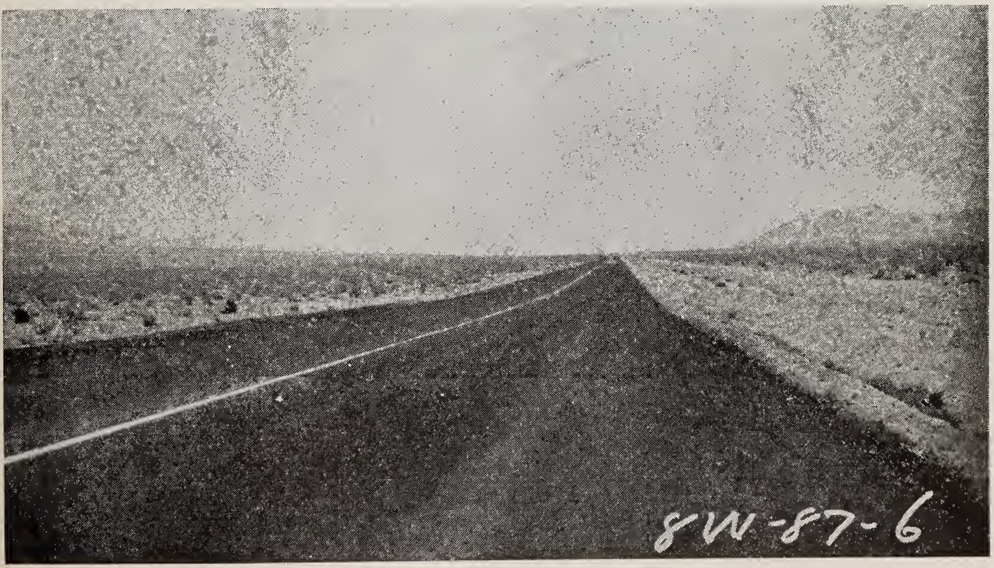
SHOULDER TREATMENT

Penetration Method



Two-Shot Penetration

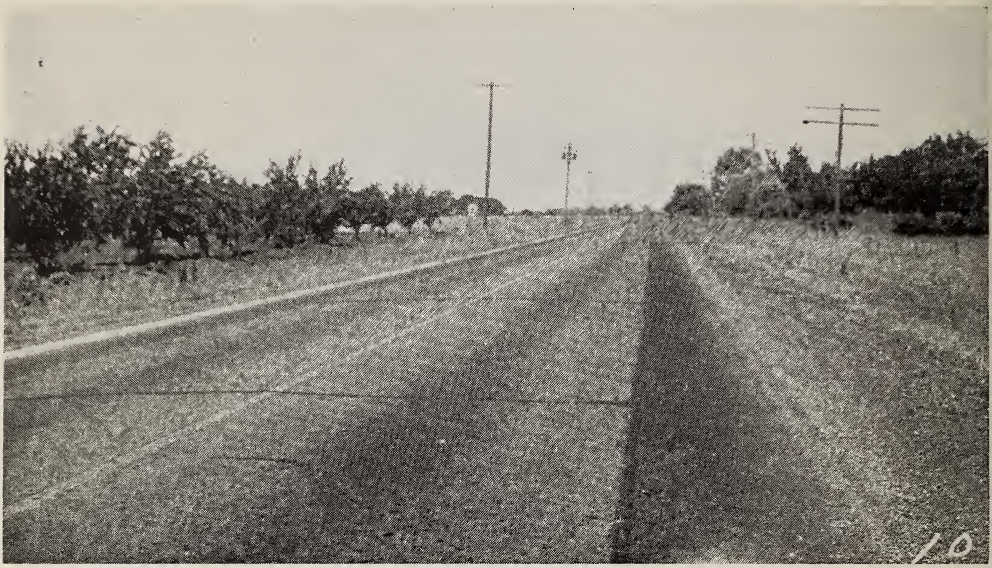
First application: ¼ gal. SC-4 — 17 lbs. ⅜"– No. 10 screenings.
Second application: 0.2 gal. SC-4 — 15 lbs. ⅜"– No. 10 screenings.



⅛ gal. SC-2 covered with sand where penetration was incomplete

SHOULDER TREATMENT (Continued)

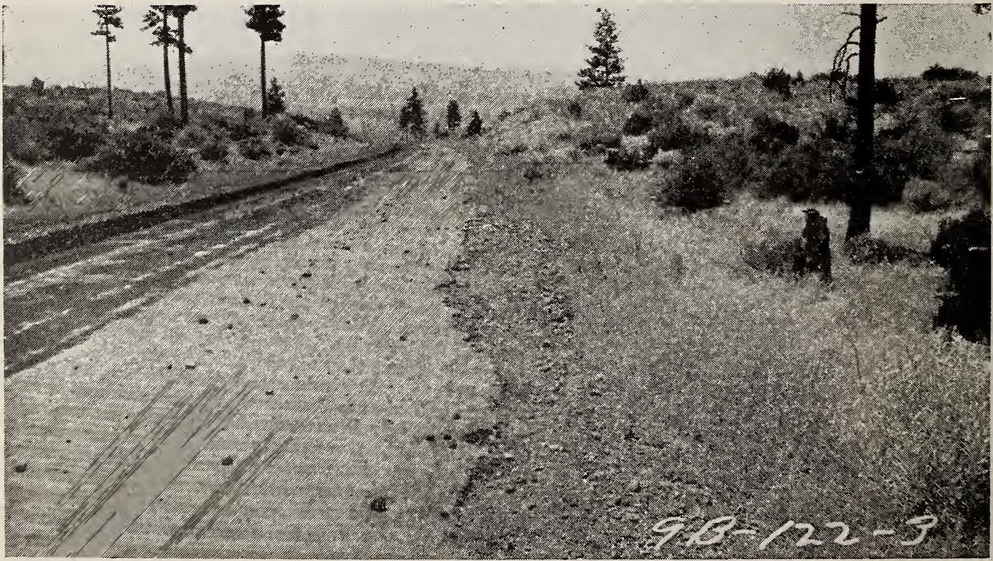
Penetration Method



Shoulders constructed of 3.5 ft. x 7 in. gravel base topped with 3 ft. x1½ in. armor coat (SC-6 and screenings)

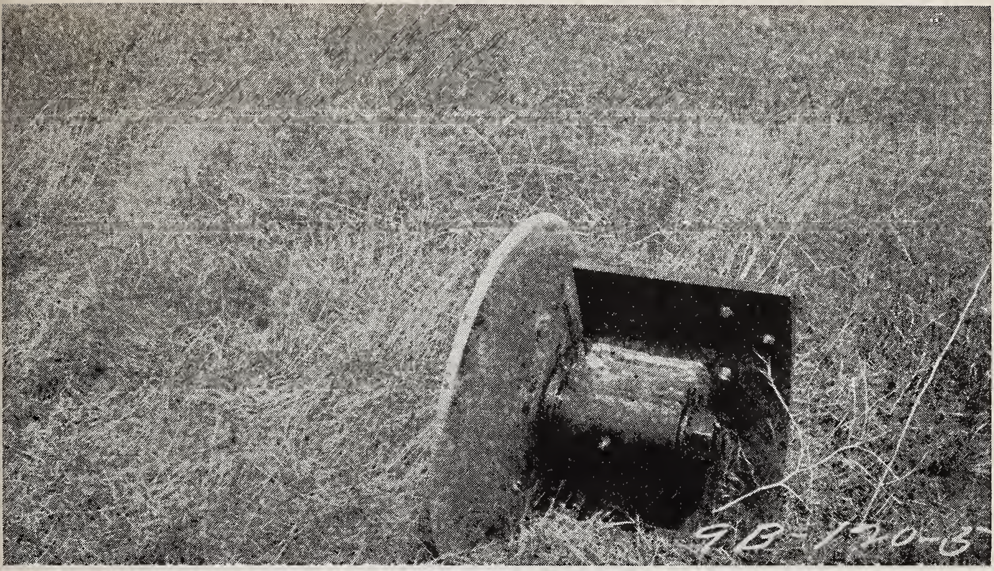
SHOULDER MAINTENANCE

Road Mixing

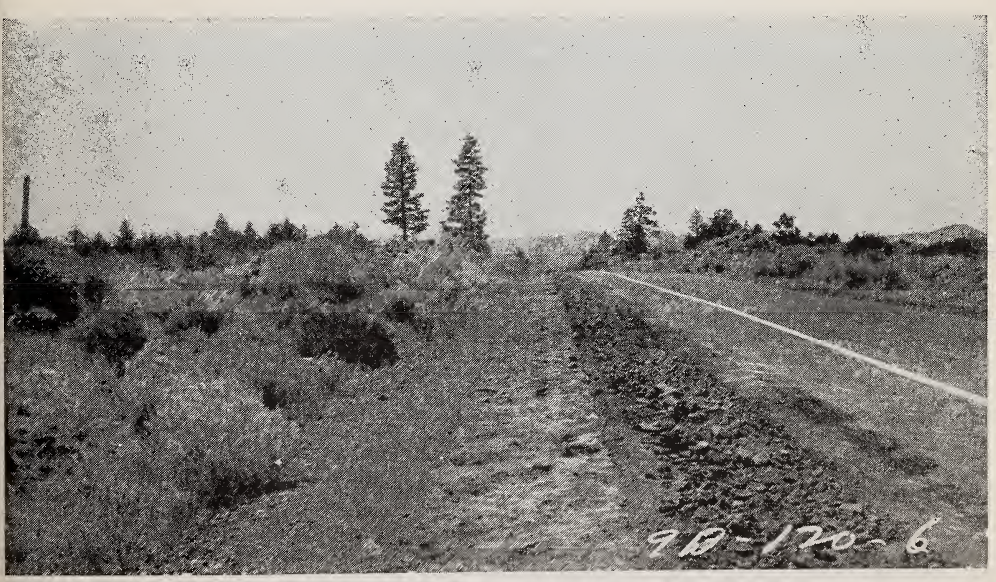


Showing disintegration edge — before treatment

Road Mixing



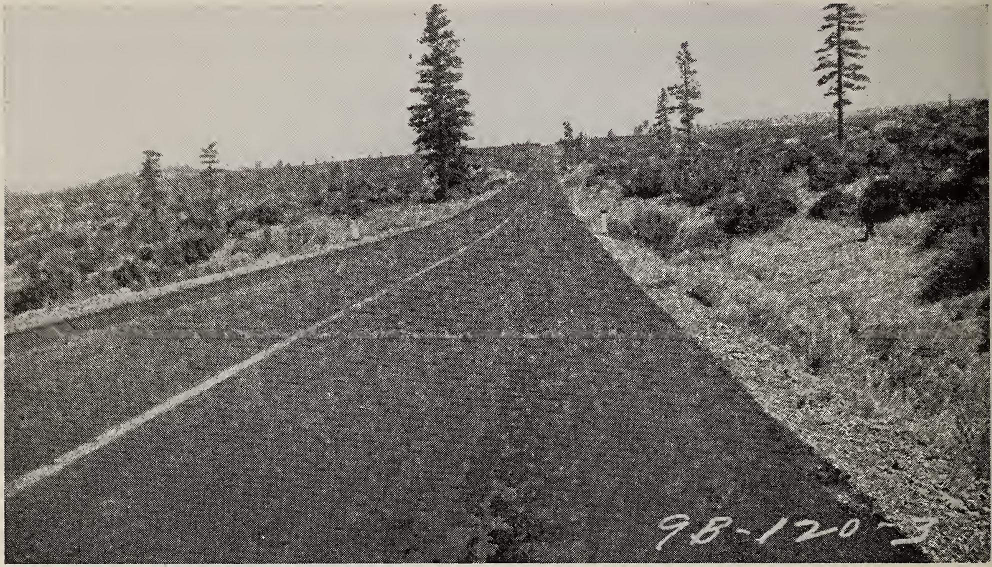
Single disc attachment to grader mold board for making clean-cut connection with surface shown in upper photo (see following photos)



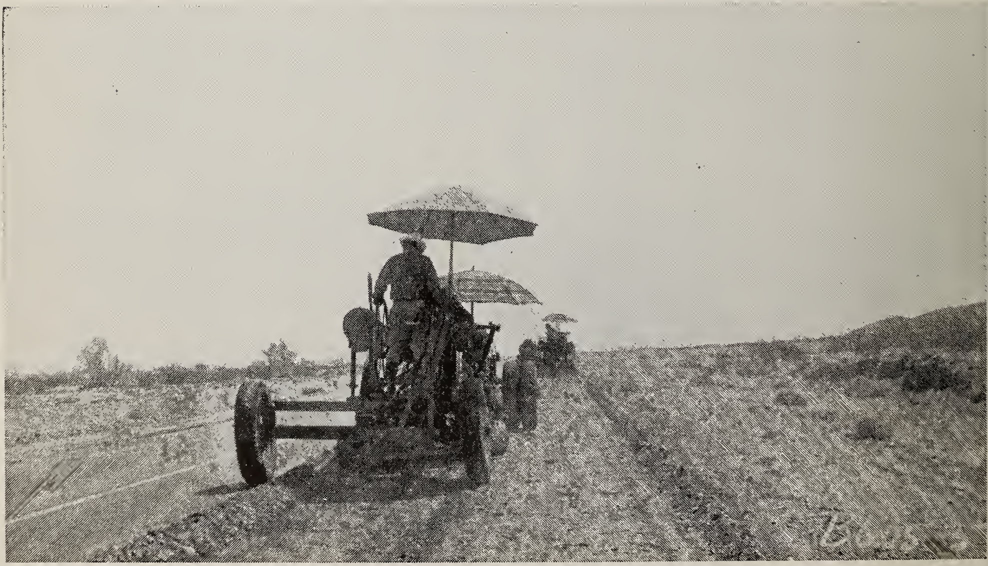
Edge has been trimmed with disc (previous photo) and scarified ready for remixing with oil

SHOULDER MAINTENANCE (Continued)

Road Mixing



Completed road mix shoulder treatment

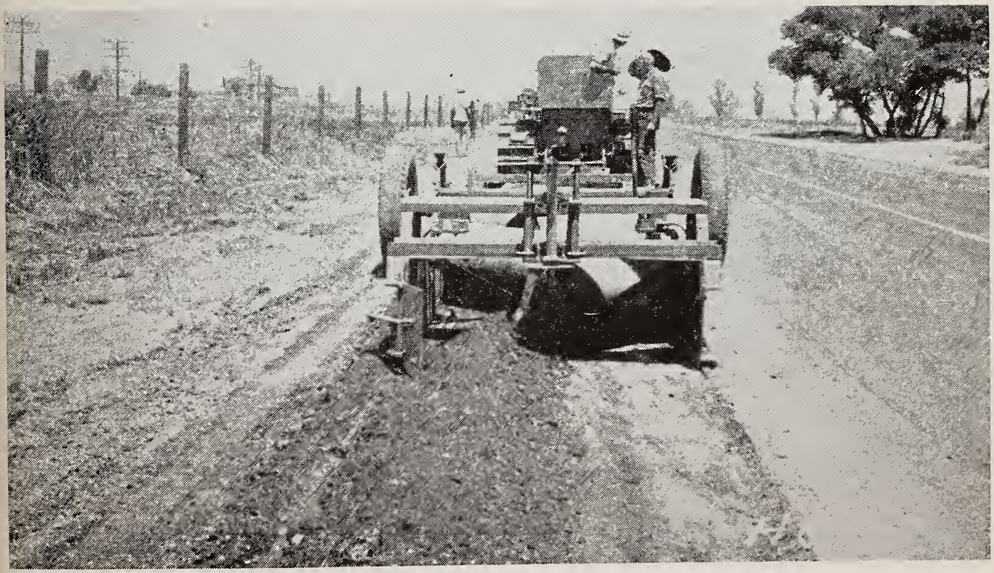


First pass of tow graders in light oil mixing project

SHOULDER MAINTENANCE (Continued)

Road Mixing

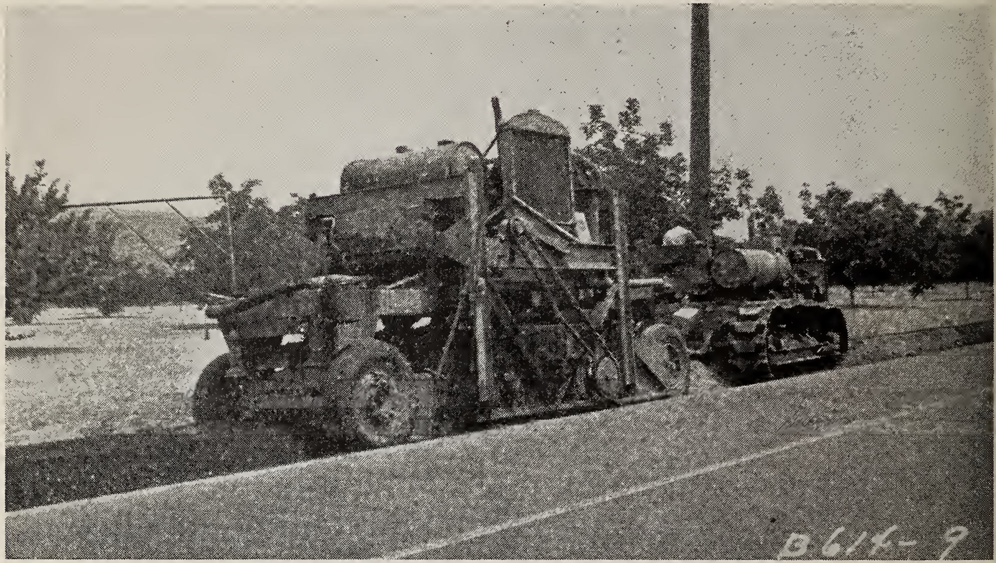
Complete light road mix — 1 gal. SC-2 oil per sq. yd.



Sled type Multiple Blade Shoulder Drag. Mixing oil shoulders.

SHOULDER MAINTENANCE (Continued)

Road Mixing

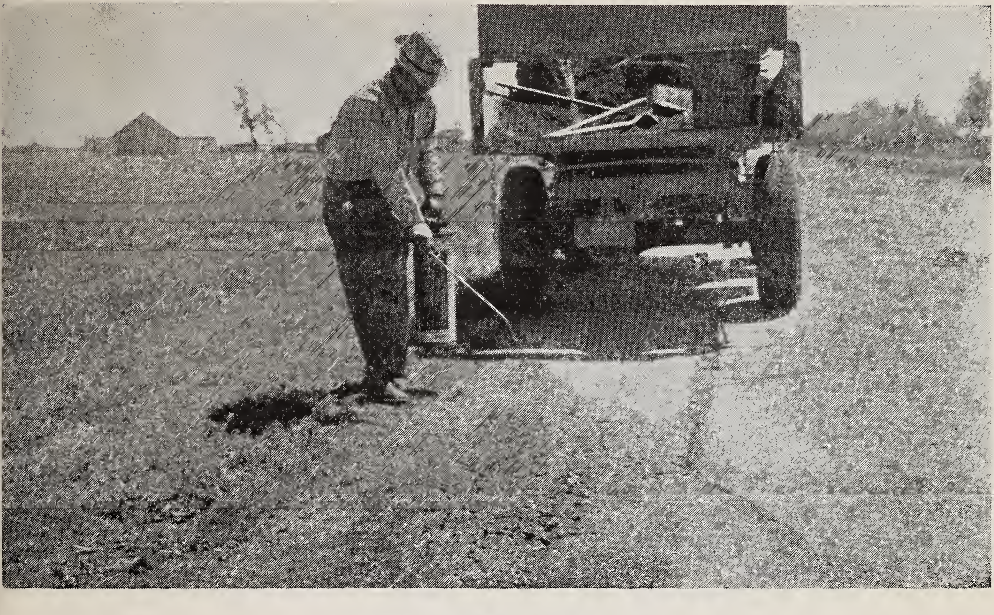


Double spool "pin type" mixer for shoulder mixing.

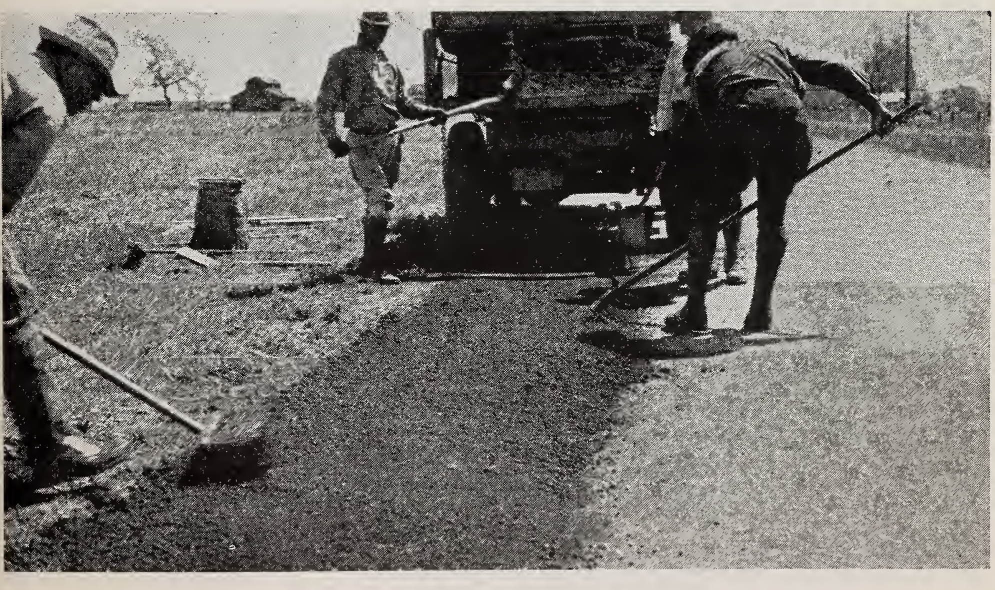
Leveling with Premix

Sweeping area to be patched

Leveling with Premix



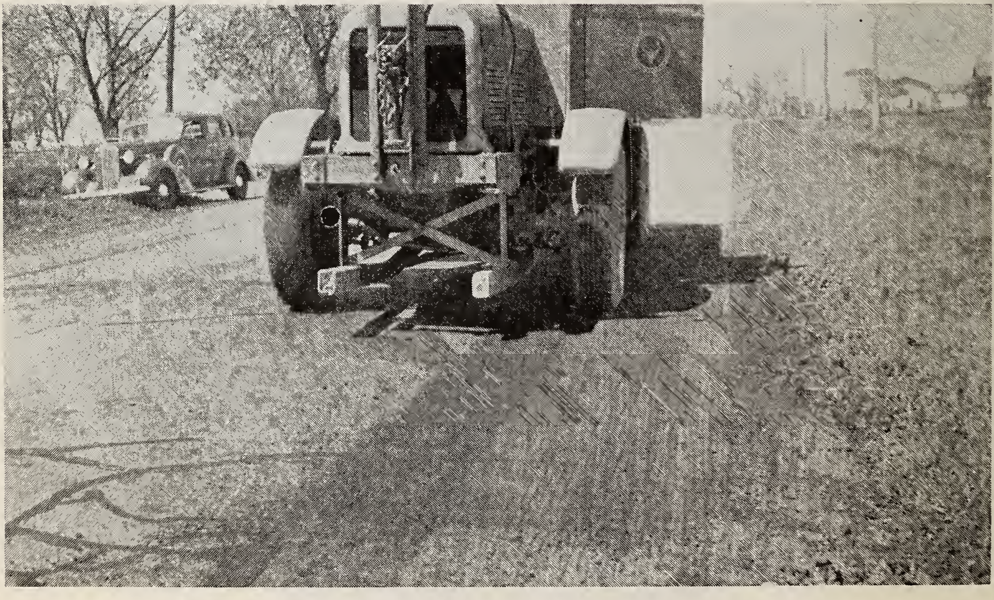
Placing emulsion tack coat



Spreading premix and hand leveling

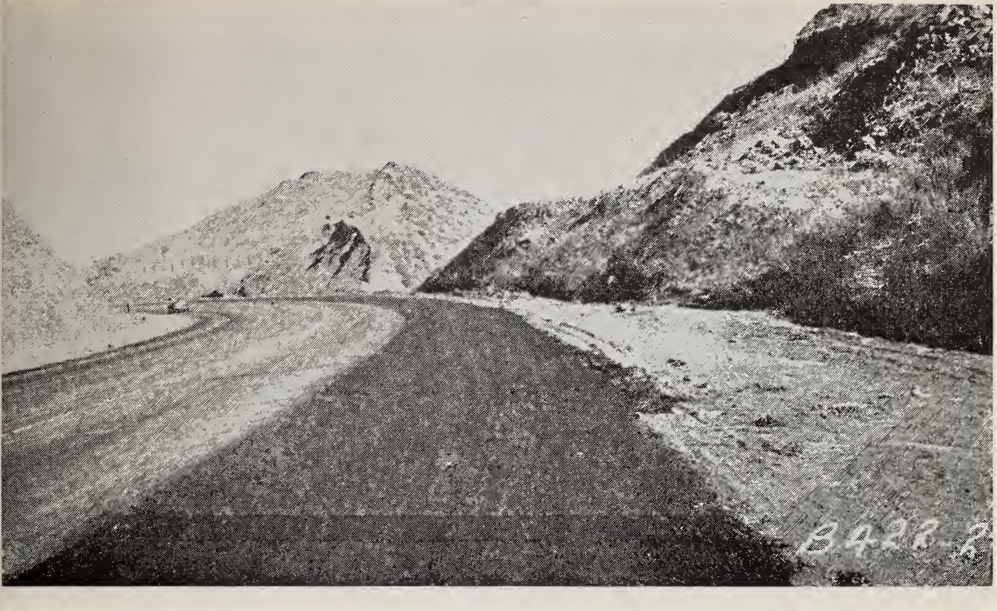
Leveling with Premix

Hand tamping

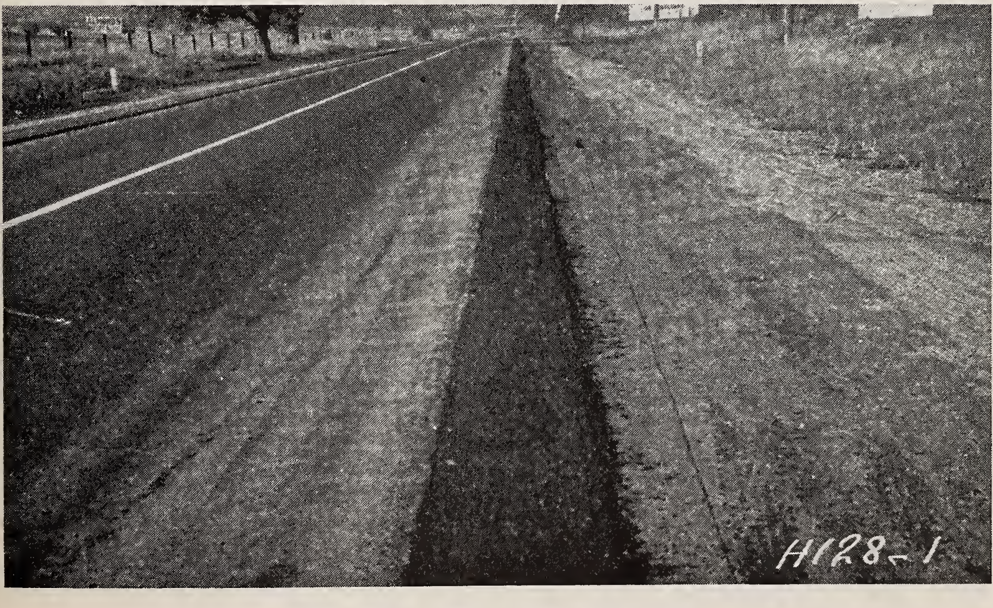


Compacting with truck wheels

PLANT MIX ON SHOULDERS



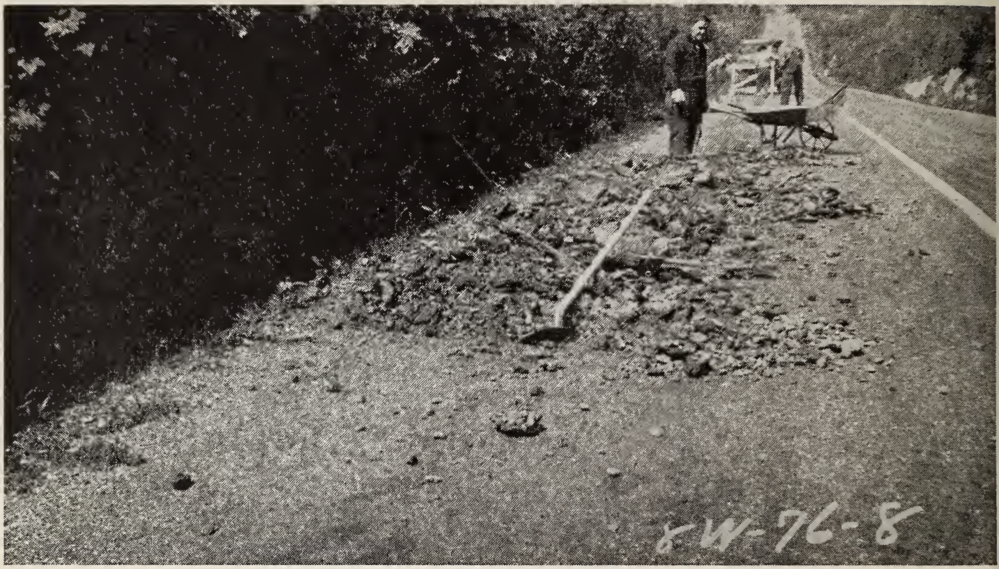
SC-4 Plant mix shoulders 1½ in. thick on macadam base.



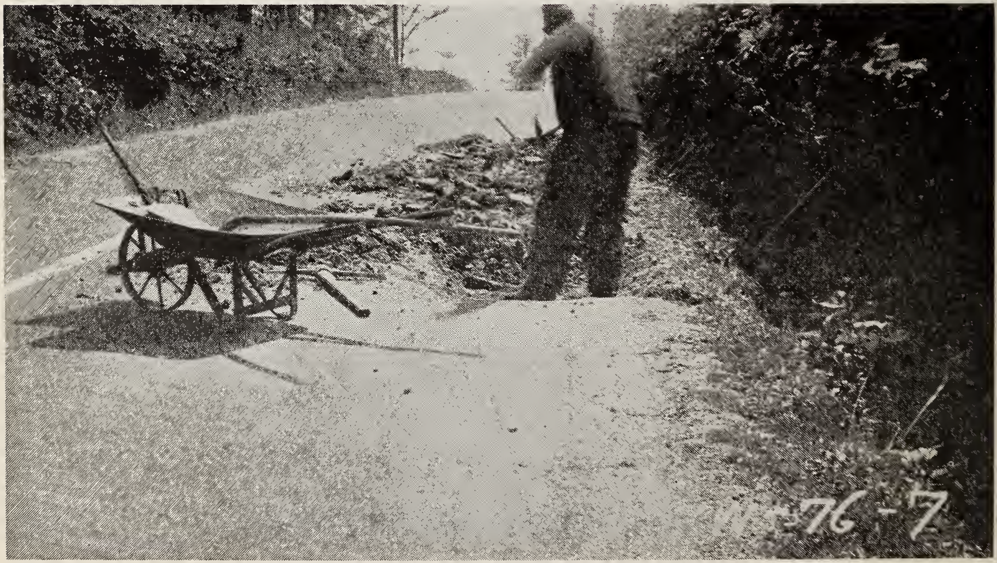
Shoulder leveling with plant mix

BASE FAILURES

Patching



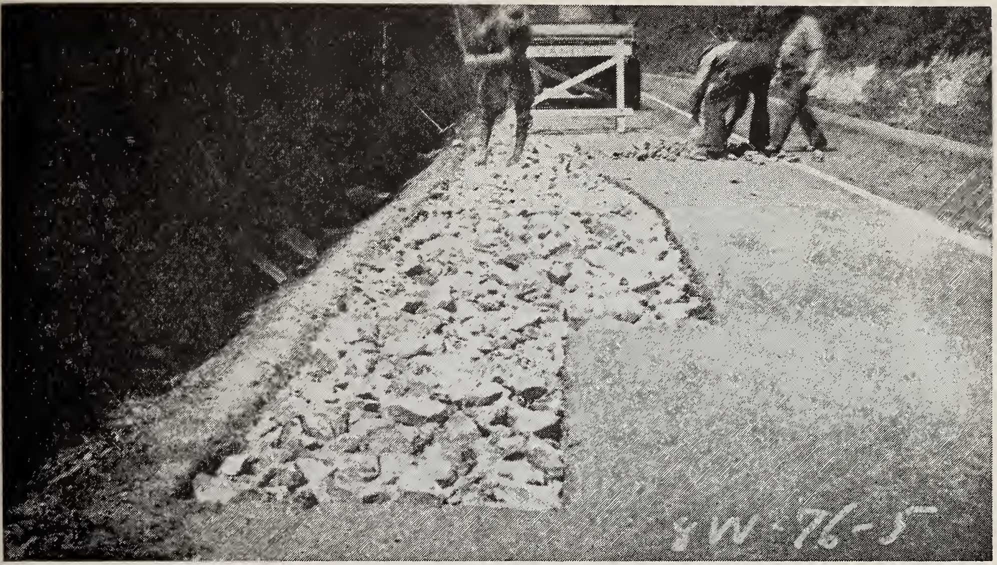
Base broken up ready for removal



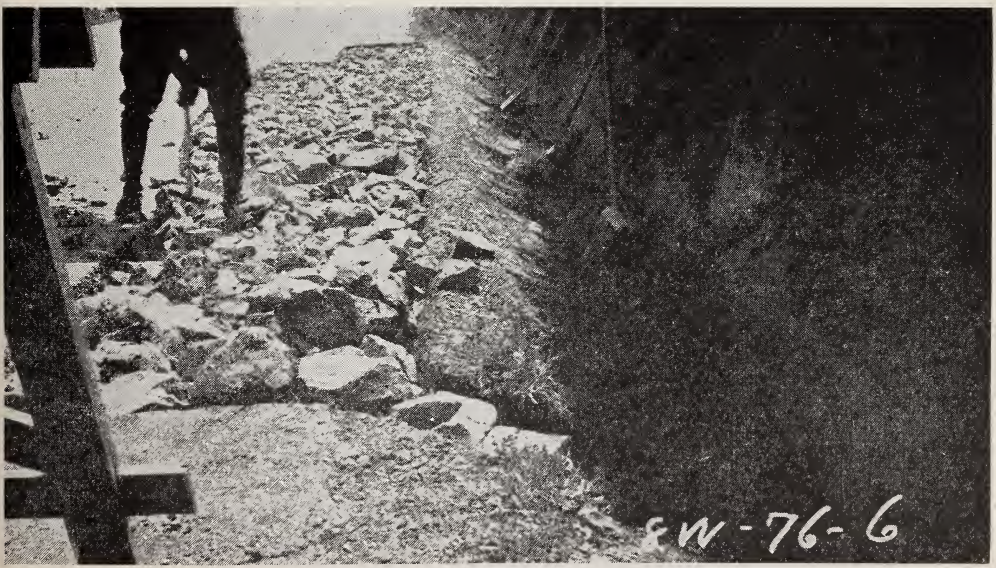
Removing unsuitable base material

BASE FAILURES (Continued)

Patching



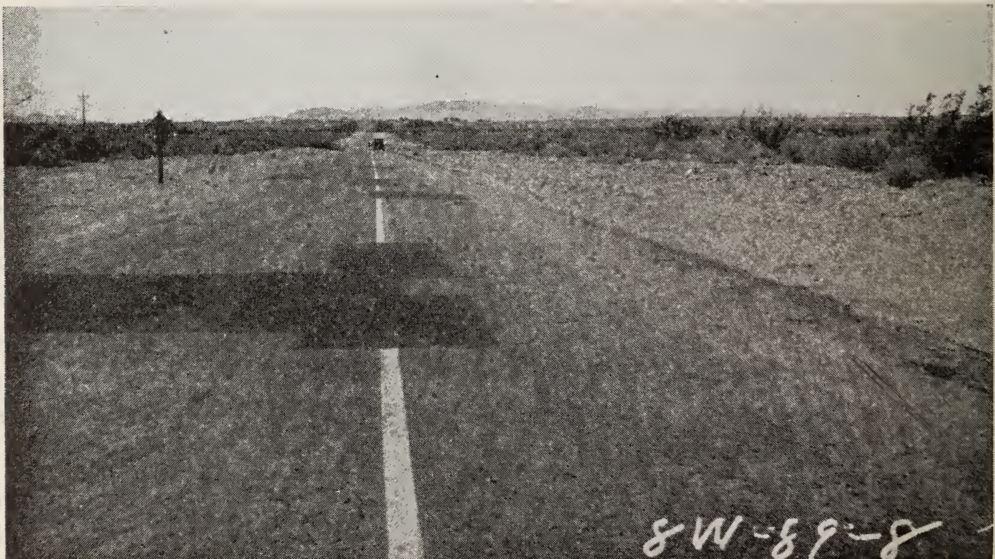
Coarse rock broken into smaller segments to form key. This particular rock was a semi-hard sandstone.



Looking up grade — note rock "bleeder" in foreground. This drains area into gutter.

Patching

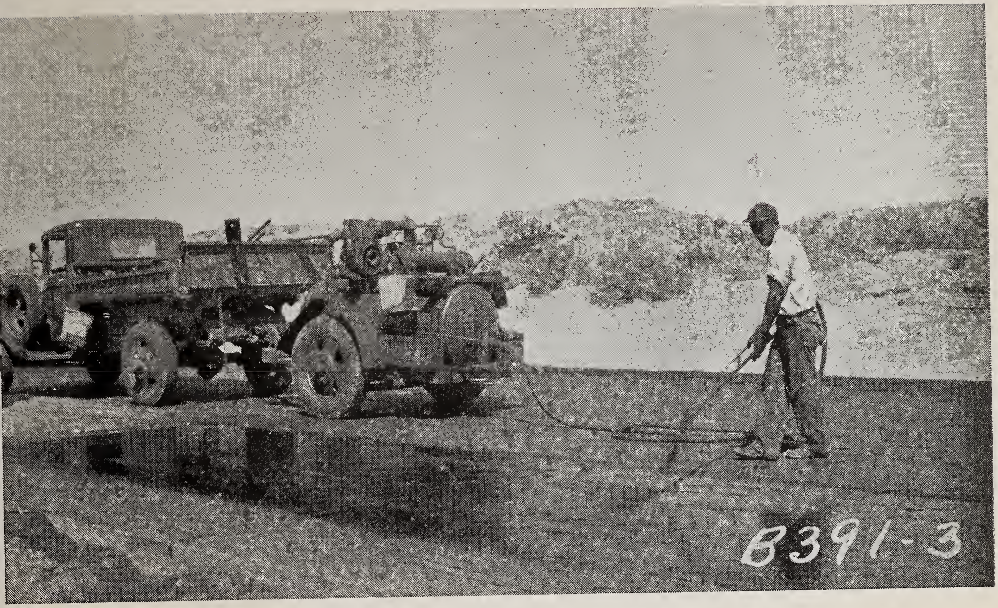
Premix patching (roadmixed local materials) on low type road.



Premix patching (roadmixed local materials) on desert road.

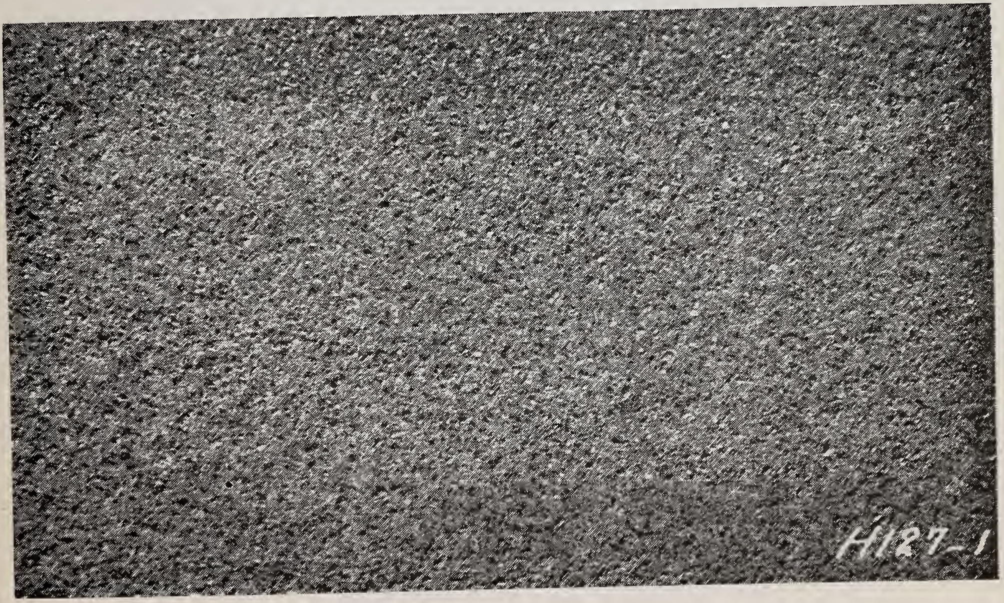
SURFACE REPAIRS (Continued)

Patching

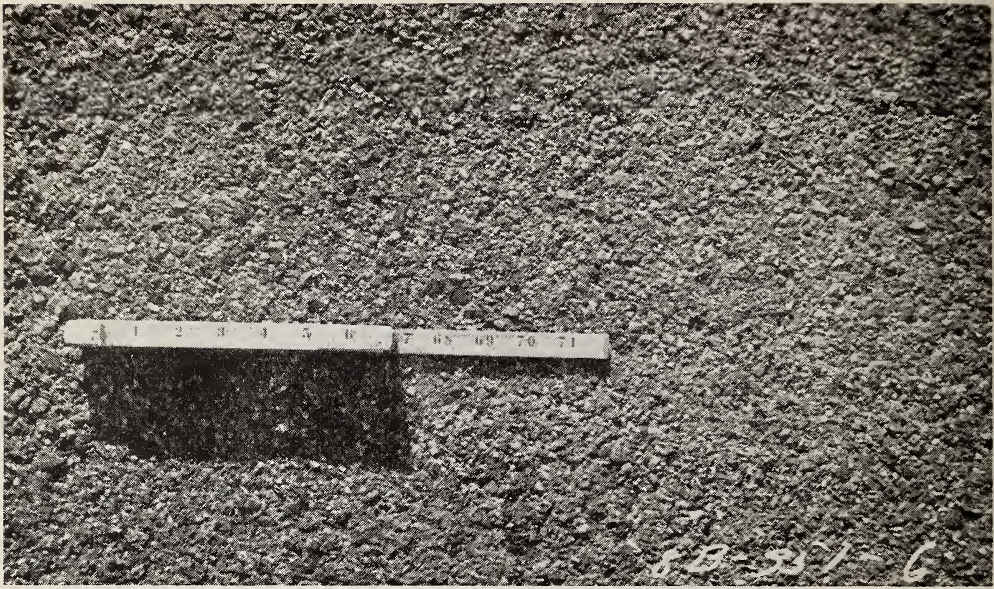


Modern 190-gallon emulsified asphalt kettle constructed by Headquarters Shop, California Division of Highways. Used for penetration and tack coat applications.

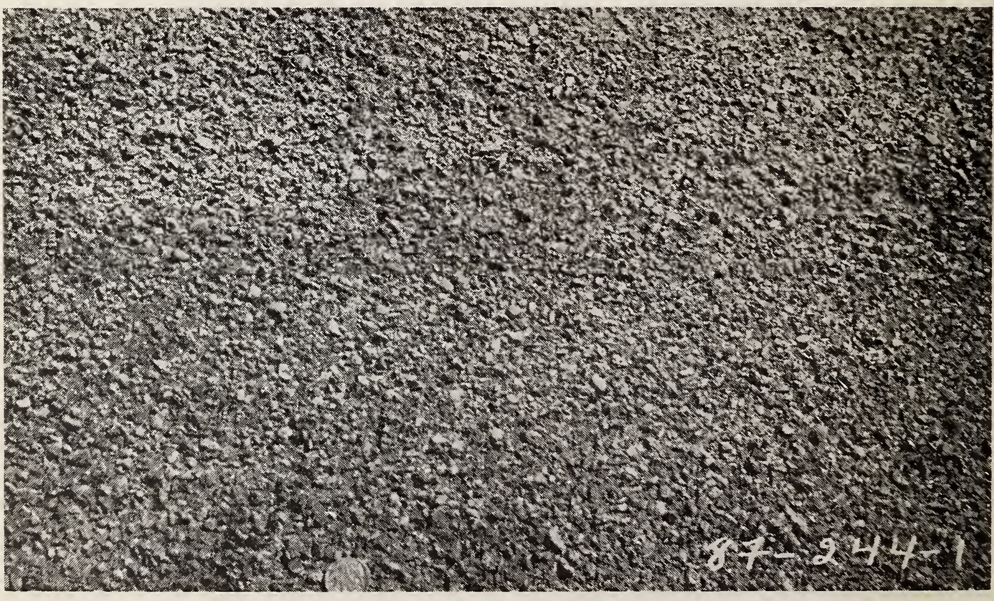
TEXTURES — NON-SKIDS AND SEALS



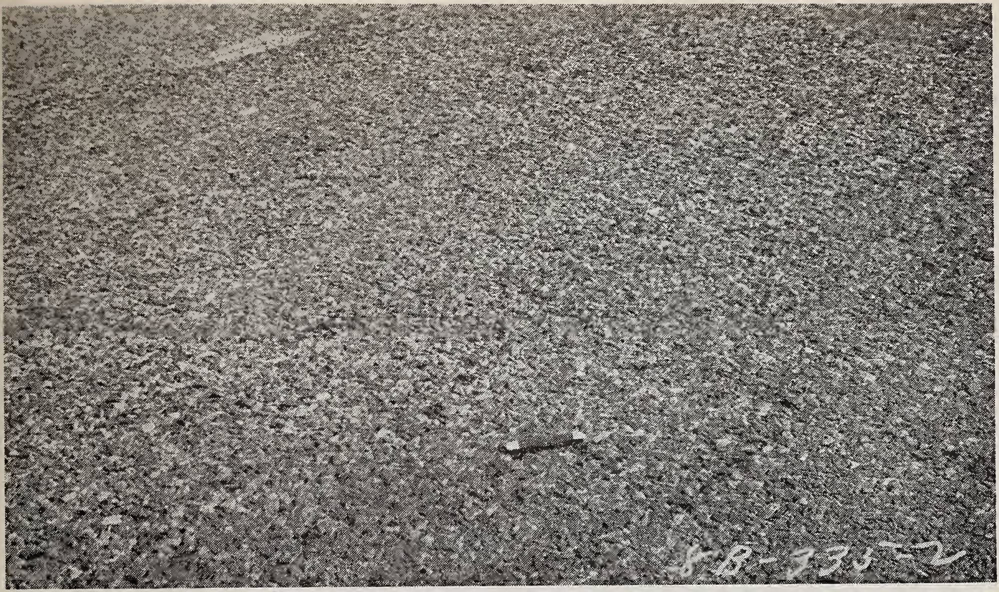
Texture of B Seal: SC-6 applied at rate of 0.17 to 0.20 gal. per sq. yd. — covered with 25 lbs. ⅜ in. - ¼ in. screenings.


1 2 3 4 5 6 7 68 69 70 71
83-321-6

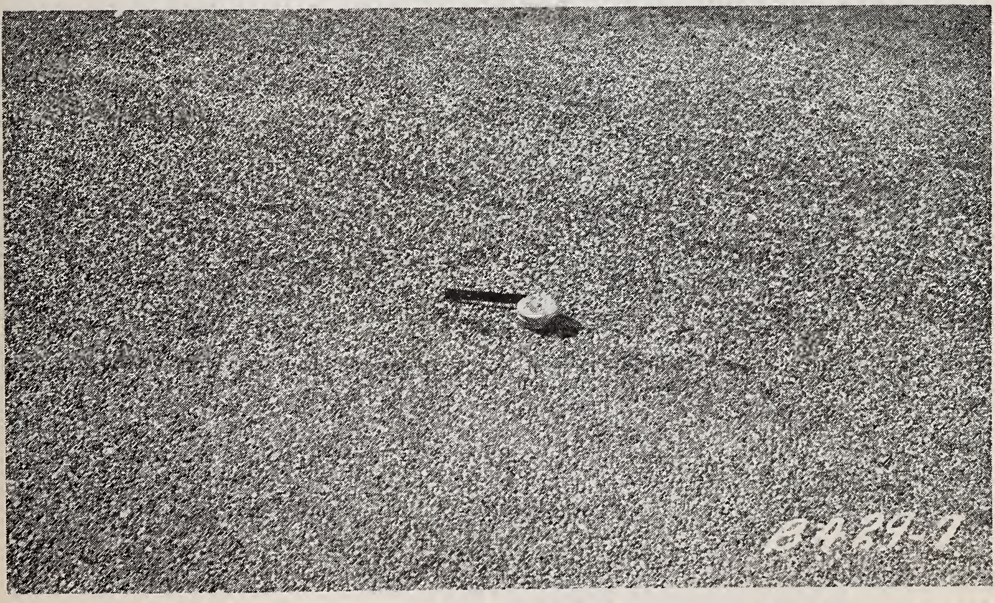
Texture of double B Seal (See Text)



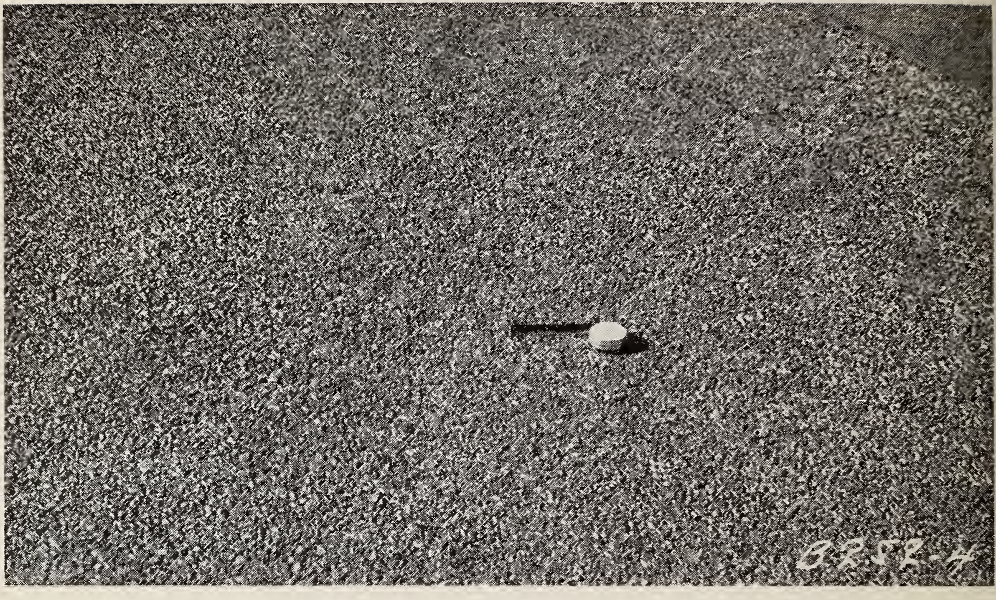
Texture of emulsion seal applied at rate of 1/6 gal. per sq. yd. and covered with 15 lbs. ¼ in. - ⅜ in. screenings.



Texture of ROMC seal (5% kerosene) applied at rate of 1/6 gal. per sq. yd. and covered with ½ in. - ¼ in. screenings.



Texture of MC-4 seal applied at rate of 0.25 gal. per sq. yd. and covered with ⅜ in. - 10 mesh round screenings.



Texture of MC-5 (wax base) seal applied at rate of 0.25 gal. per sq. yd. and covered with ½ in. - ⅛ in. screenings.

NON-SKID TEXTURE — PLANT MIX BLANKET

Surface texture of SC type plant mix

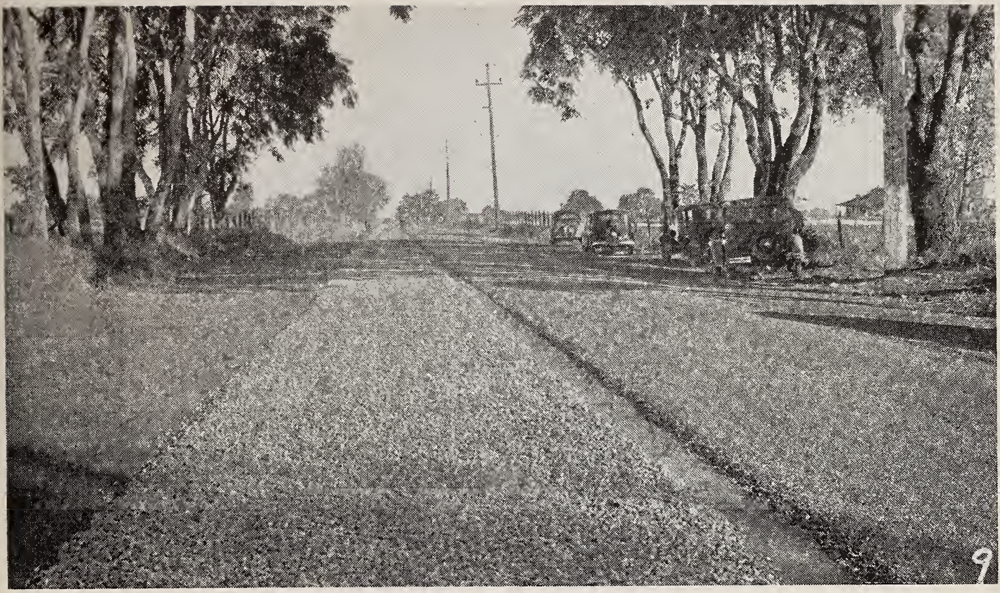
Mineral Aggregate graded as follows:

Passing ⅜ in. screen	100%
Passing 3 mesh sieve (0.263)	90% to 100%
Passing 10 mesh sieve	50% to 60%
Passing 40 mesh sieve	20% to 30%
Passing 80 mesh sieve	13% to 18%
Passing 200 mesh sieve	8% to 11%

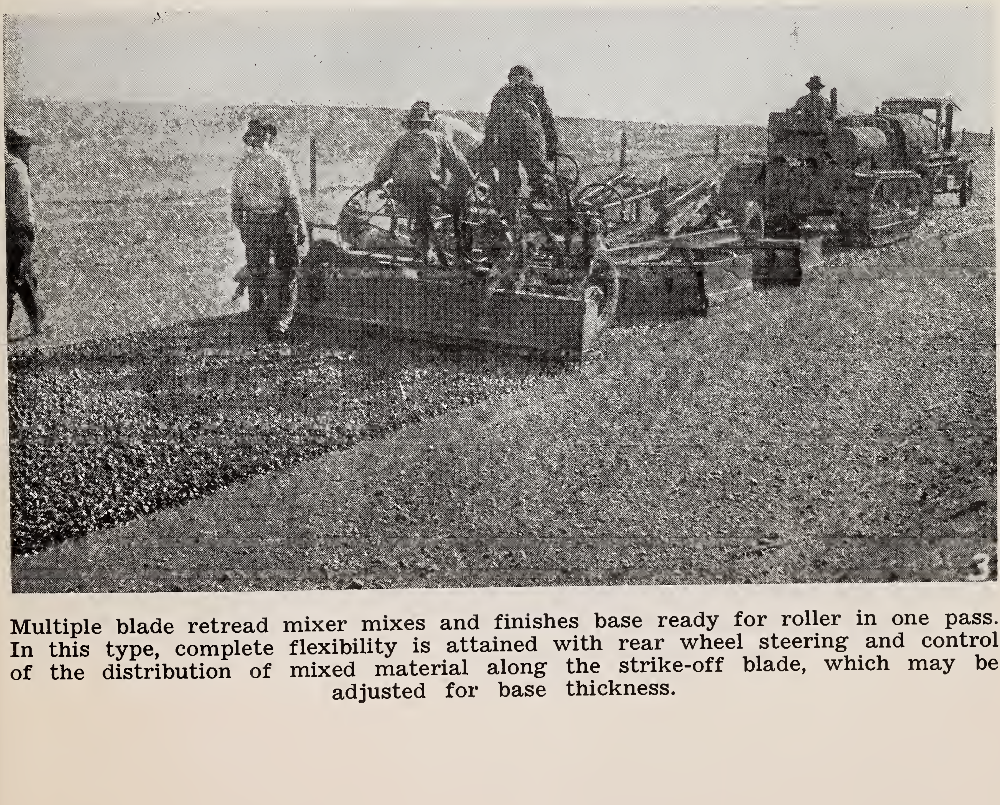
Specifications permit variation of 4% to 5.5% of liquid asphalt (SC-4).

RETREADS

Emulsion

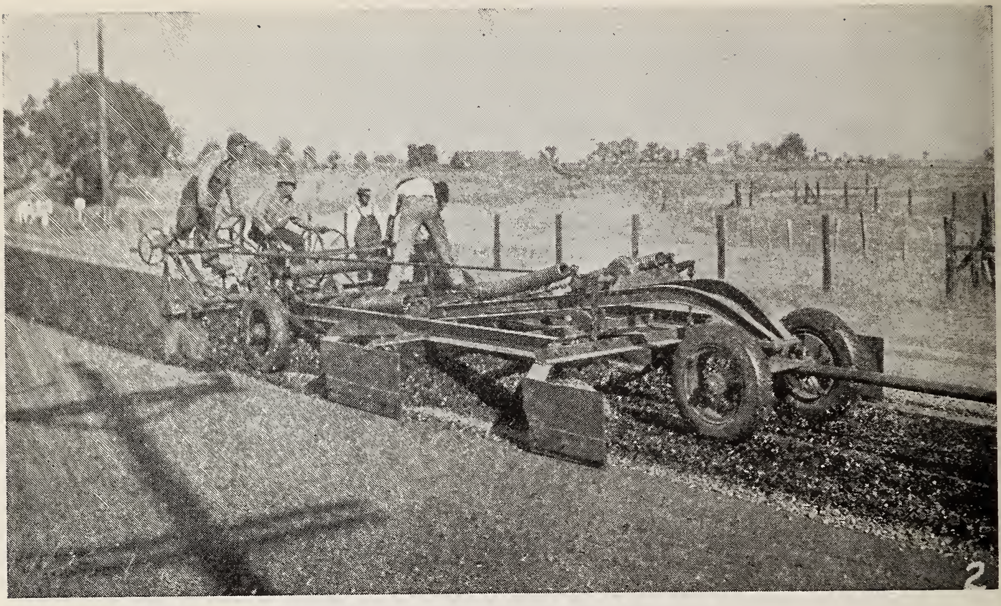


Right half completed; base rock laid out on left ready for application of mixing type emulsion.

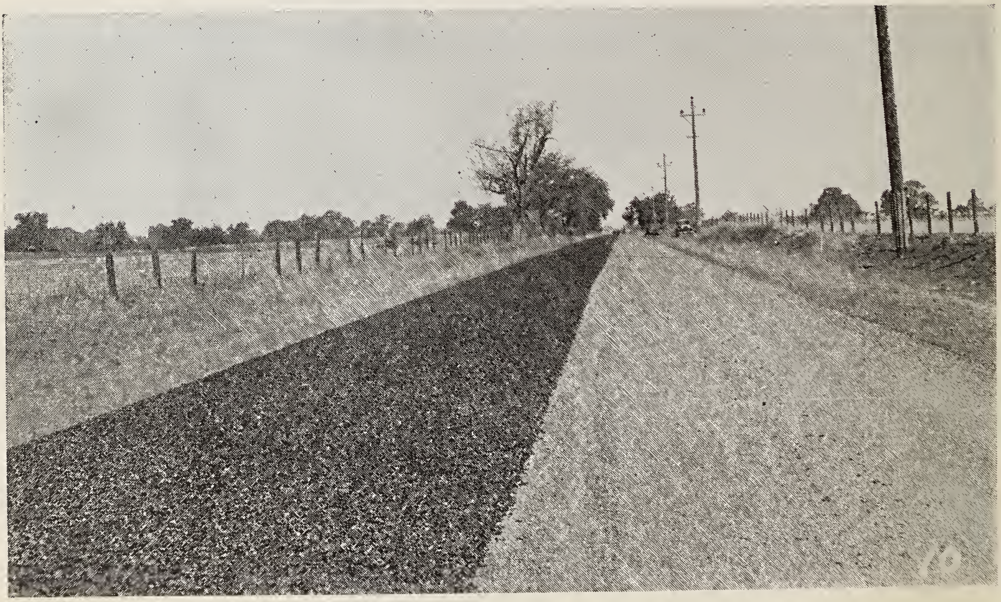


Multiple blade retread mixer mixes and finishes base ready for roller in one pass. In this type, complete flexibility is attained with rear wheel steering and control of the distribution of mixed material along the strike-off blade, which may be adjusted for base thickness.

RETREAD (Continued)



Front view of retread mixer

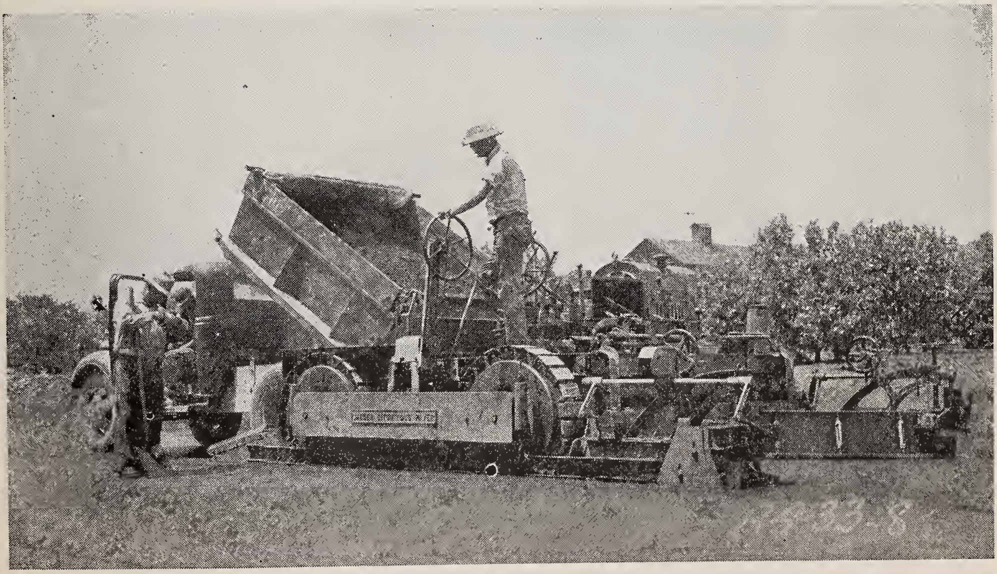


View of base rock mixed and laid in one pass of retread mixer. Rolling is deferred until emulsion has passed the "stripping" stage.

Texture of finished emulsion retread surfacing

BLANKETS

Plant Mix



Plant mix blanket spread finished and tamped with mechanical paver designed for use without header boards

Plant Mix

Rear view showing plant mix laid and ready for roller.

Spreading plant mix for 1 in. blanket on one side which has already received tack coat application

BLANKETS (Continued)

Plant Mix

Placing light tack coat on other side.

Finished plant mix blanket, 1 inch thick. The original pavement vialoged 50 units per mile which was reduced to 15 units of roughness with the placing of the blanket.

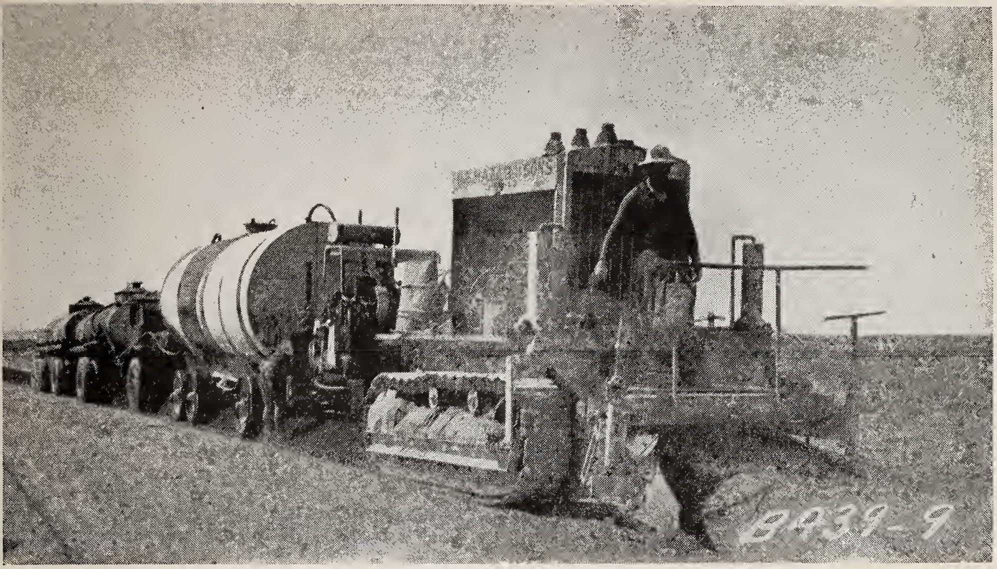
BLANKETS (Continued)
Plant Mix

Portable type power roller capable of being towed. Water ballast is drained, roller is jacked clear of ground and entire roller is supported on rear pneumatic tired wheels and tow bar.

MECHANICAL MIXERS

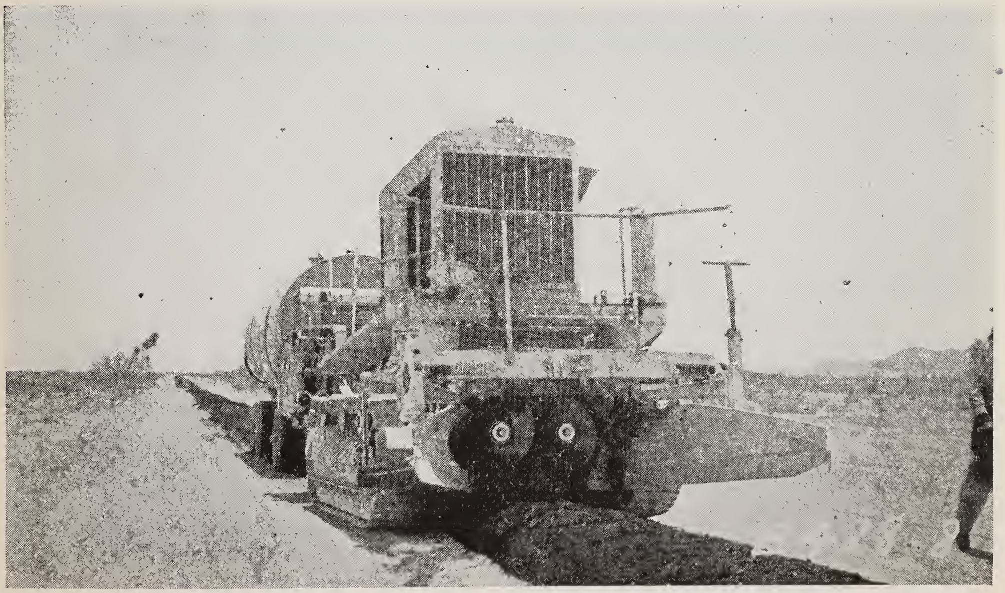
Pugmill type mixer on crawler treads

MECHANICAL MIXERS (Continued)



Pugmill type mixer -— another view

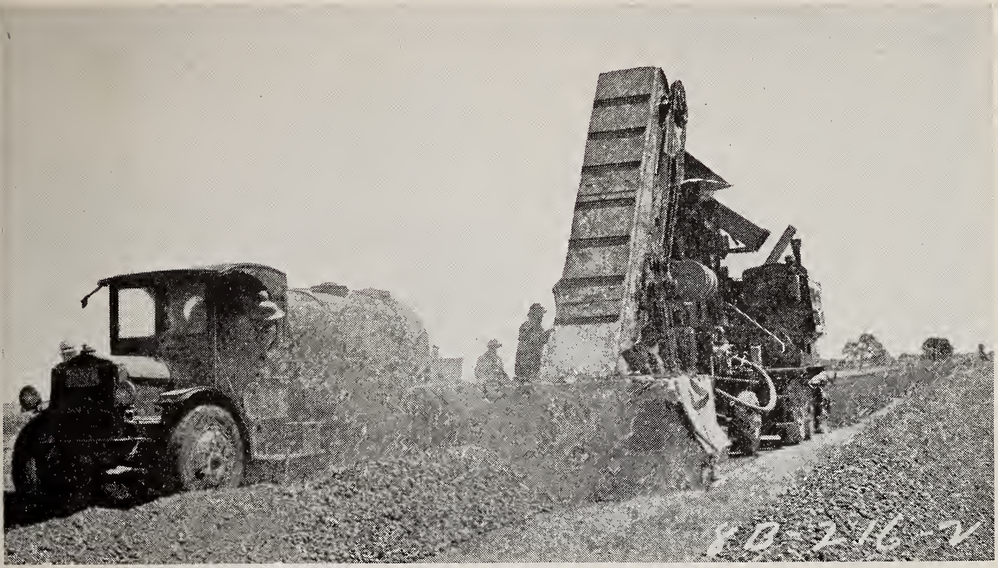
TRAVELING MIXERS



Pugmill type mixer with front end raised

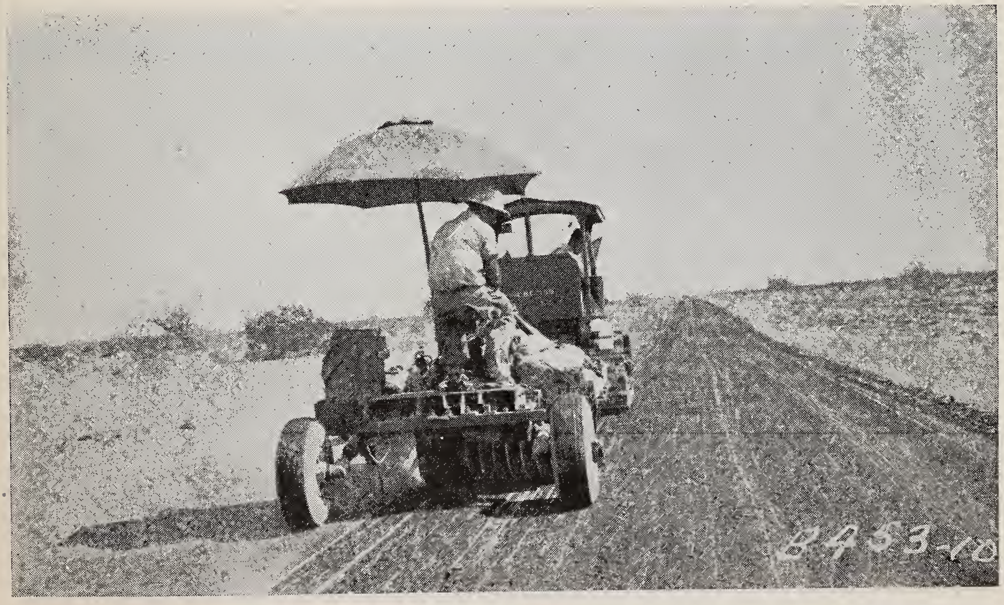
Traveling mixing machine with oil supply taken from tank truck, mixing base with stabilizing emulsion

Rear view of traveling mixer — emulsion stabilized base — note wet consistency of finished mix deposited in windrow.



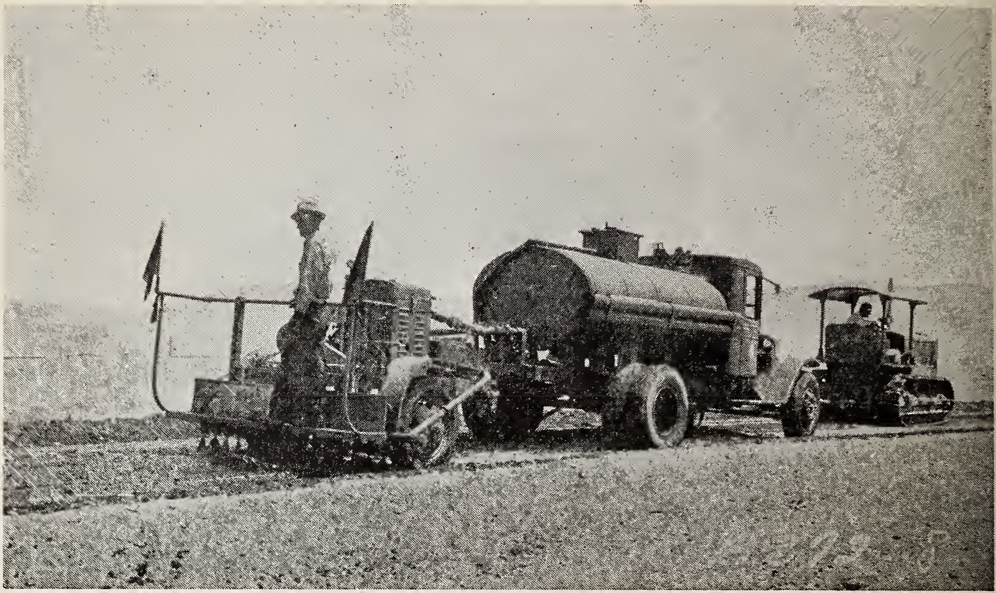
Front view showing pickup of dry aggregate in windrow and spreading mixed material in rear

EQUIPMENT FOR RETREATMENT



Disc scarifier equipped with low pressure tires and hydraulic control.

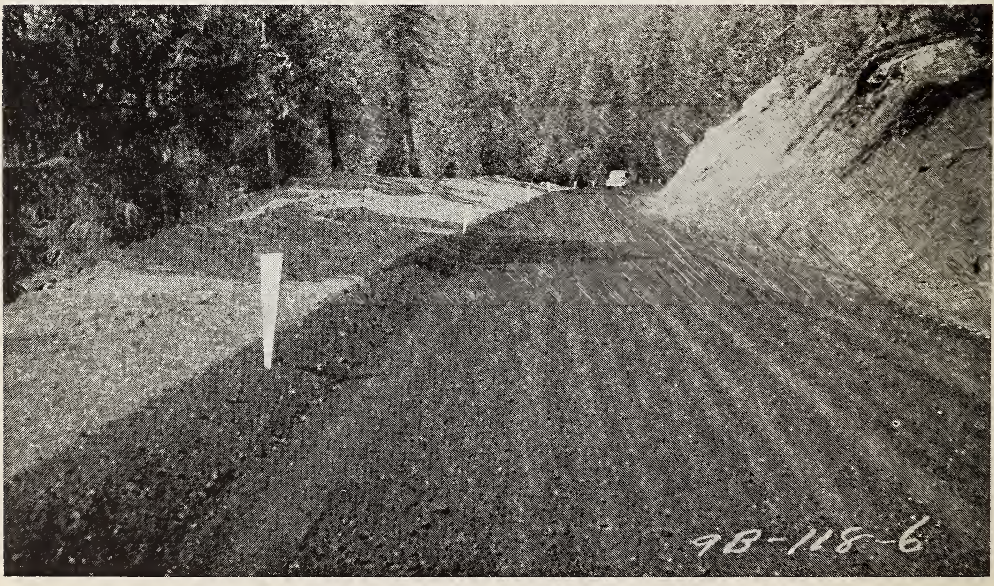
EQUIPMENT FOR RETREATMENT (Continued)



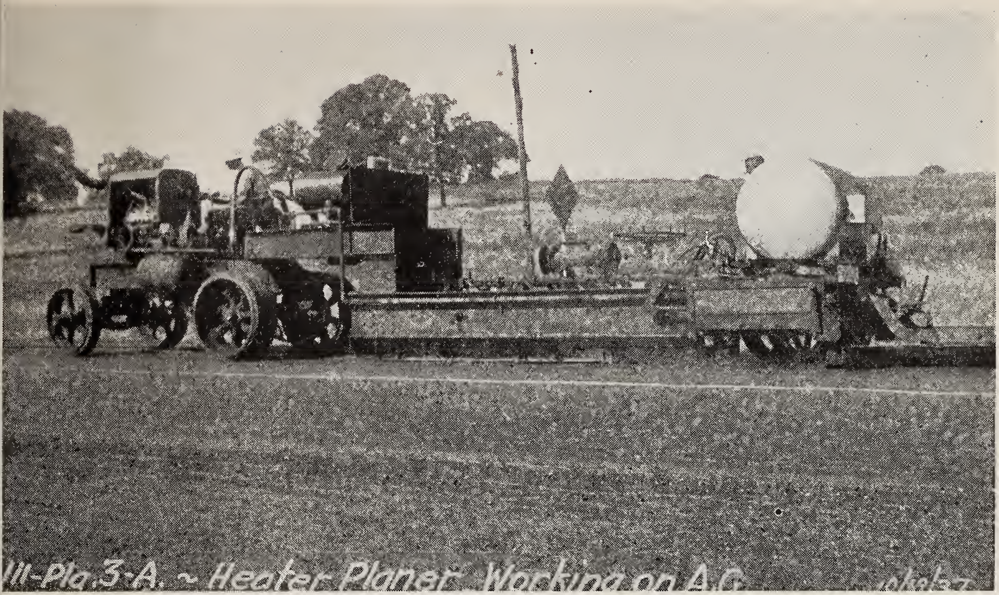
Trailer type distributor utilizing "booster" truck in applying SC liquid asphalt for road mixing

REPROCESSING

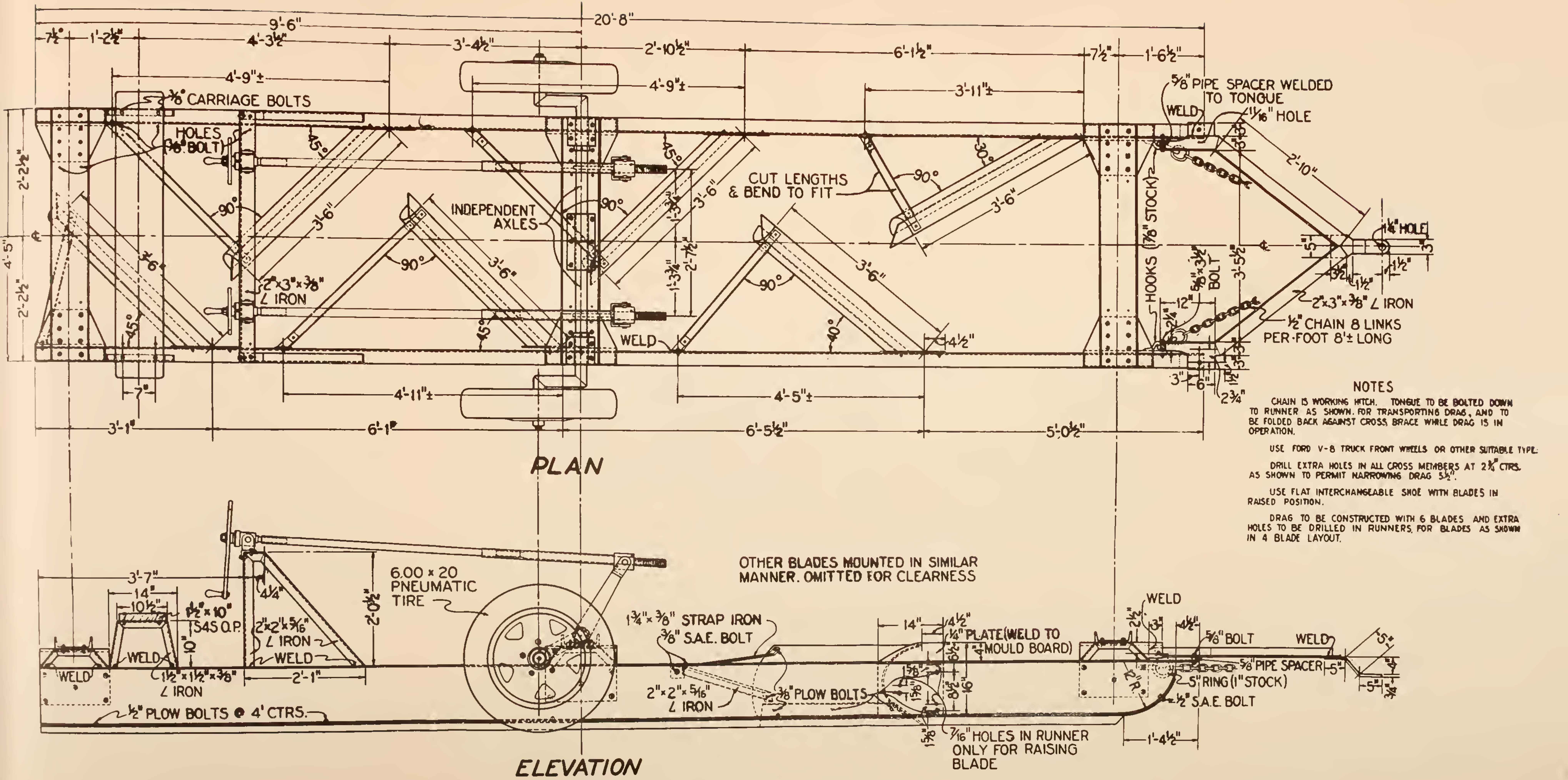
Road Mixing



Road mixed reprocessing with ROMC-3 added at the rate of ⅜ gallon per square yard. Note wedges placed to protect traffic at night.


III-Pla.3-A. ~ Heater Planer Working on A.C.

Heater planer used to remove rich spots and smooth up surface prior to placing non-skid treatment


20'-8"
9'-6"
7½"
1'-2½"
4'-3½"
3'-4½"
2'-10½"
6'-1½"
7½"
1'-6½"
4'-9"±
4'-9"±
3'-11"±
⅜" CARRIAGE BOLTS
HOLES (⅜" BOLT)
⅝" PIPE SPACER WELDED TO TONGUE
WELD
1/16" HOLE
2'-10"
45°
3'-6"
CUT LENGTHS & BEND TO FIT
90°
30°
3'-6"
INDEPENDENT AXLES
90°
90°
1-3¾"
2'-7½"
1-3¾"
3'-6"
90°
3'-6"
3'-6"
90°
3'-6"
2"×3"×⅜" ∠ IRON
HOOKS (⅞" STOCK)
⅝"×3½" BOLT
3'-5½"
1¼" HOLE
5"
3"
1½"
1½"
3½"
2"×3"×⅜" ∠ IRON
½" CHAIN 8 LINKS PER·FOOT 8'± LONG
12"
2¼"
45°
45°
40°
WELD
4½"
3"
6"
3"
2¾"
2'-2½"
2'-2½"
4'-5"
7"
4'-11"±
4'-5"±
3'-1"
6'-1"
6'-5½"
5'-0½"
PLAN
OTHER BLADES MOUNTED IN SIMILAR MANNER. OMITTED FOR CLEARNESS
3'-7"
14"
10½"
1½"×10" S4S O.P.
4¼"
2"×2"×5/16" ∠ IRON
WELD
6.00 × 20 PNEUMATIC TIRE
2'-0½"
10"
WELD
WELD
1½"×1½"×⅜" ∠ IRON
2'-1"
½" PLOW BOLTS @ 4' CTRS.
1¾"×⅜" STRAP IRON
⅜" S.A.E. BOLT
2"×2"×5/16" ∠ IRON
⅜" PLOW BOLTS
14"
4½"
¼" PLATE (WELD TO MOULD BOARD)
7/16" HOLES IN RUNNER ONLY FOR RAISING BLADE
WELD
2¼"
3"
4½"
⅝" BOLT
WELD
⅝" PIPE SPACER
5" RING (1" STOCK)
½" S.A.E. BOLT
12" R
1'-4½"
5"
5"
ELEVATION

NOTES

CHAIN IS WORKING HITCH. TONGUE TO BE BOLTED DOWN TO RUNNER AS SHOWN. FOR TRANSPORTING DRAG, AND TO BE FOLDED BACK AGAINST CROSS BRACE WHILE DRAG IS IN OPERATION.

USE FORD V-8 TRUCK FRONT WHEELS OR OTHER SUITABLE TYPE.

DRILL EXTRA HOLES IN ALL CROSS MEMBERS AT 2¼" CTRS. AS SHOWN TO PERMIT NARROWING DRAG 5½".

USE FLAT INTERCHANGEABLE SHOE WITH BLADES IN RAISED POSITION.

DRAG TO BE CONSTRUCTED WITH 6 BLADES AND EXTRA HOLES TO BE DRILLED IN RUNNERS, FOR BLADES AS SHOWN IN 4 BLADE LAYOUT.

Sub-Base Repair Practices in Montana

By

S. B. SANDERS
Division Engineer
Montana Highway Department

During the dim pioneer age of highway construction in Montana, when its present commodius system of 5,320 miles, of which 4,110 have bituminous treated surface, was in swaddling clothes and the highway department was just attaining an aspect of pubescence, bituminous treatment of highway surfaces in our state was merely a dream of the thinnest vapor — a subject of discussion for only the fanatical-minded highway engineers. Currently, the motorist hunting pack was crying with increasing crescendo, "Get us out of the mud!"

In that space of archaic eternity, struggling from the highway mire was accomplished, fundamentally, much as it is now, but the acme of a highway surface—a ribbon of crushed gravel—was easier accessible, quicker to invite the motorist's confidence, and sustain longer his good will than the dustless surface of today. Road perfection to him was spelled out through his ability to traverse contrastingly short distances without the supplemental tractive force of a farm team being applied to the pride of the family fleet in extricating it from some of the numerous formidable bog holes.

So, with resolute integrity and single purpose, but calmly unperceptive of the complexities of bituminous treatment, which our methods forebode, we gravelled on and on; soon the mud was turned to rock dust. But a new theme song of a strengthened pack, "Give us black top!" was ascending higher and higher on the familiar tune.

Even then, theory of the prescribed method of placing base course and top course gravel was hypothetical and the writer has beheld highway engineers, of normally complacent behavior under the most irrepressible conditions, instantaneously converted into veritable raving maniacs when a contractor inadvertently or wittingly desecrated the chastity of the gravel semi-trench design.

The semi-trench section was constructed through the simple expedient of a blade grader cutting a trench from two to four inches in depth from fourteen to eighteen feet in width, on finished subgrades of twenty to twenty-six feet wide. The excavation from the trench was bladed in two equally divided windrows placed immediately outside the trench line toward the shoulder, later to be used in part, if required (and qualitatively acceptable) for binder, the remainder levelled off to the shoulder preceding placing of the top course. In the trench was deposited the subbase gravel, normally in compacted depths of four to six inches. Standard base gravel was crushed to 1½″ or 1¼″ maximum retaining from forty to sixty per cent passing the ¼″ round. On the completed base course was placed four to six inches of crushed top course gravel, ¾″ maximum retaining forty to sixty-five per cent passing the ¼″ round, a foot overlapping each edge of the trench and feathered out from that point to the shoulder.

Conception calculated the trench to produce unitive influence capable of retaining intact the consolidated metal and moisture in a resistive structure. Traffic in those days was not of a volume sufficient to exert a combination with natural enemies in comprehending for us a prediction of the ravaging results to be expected from this manner of subbase construction when the surface was subjected to bituminous treatment, but the prime destructive forces, capillary and surface moisture with which we are now more familiar, were in subtle attendance.

As ever, it is not now difficult to perceive and we are amazed at our lack of intuitiveness when the problem of redesign and repair become firmly established in our minds and subsequent construction methods substantially revised. When the pack's incessant clamoring provoked a premature large scale program of bituminous surface treatment, a considerable mileage of this type was oiled without previous advantage of research or experiment, and the incompetencies of the semi-trench as an able support to bituminous surfaces were only gradual in becoming apparent. It was, however, soon evident that the trench was a moisture trap capable of generating most detrimental influences. Failures were most glaring along the trench's edge, the conversion point of moisture and naturally, with the advent of center striping, most vulnerable for traffic. That portion of the mat extending beyond the trench devoid of base gravel support was, except in those instances of exceptionally favorable soil types, practically a total loss toward sustaining traffic.

Precisely, the problem at hand was to eliminate the trench, provide the subgrade with full width and substantial thickness of base gravel, and by the same token, salvage the oil mat aggregate, which was much too valuable to waste or incorporate with the base course. It is not to be construed that wholesale mileages of this type suffered complete failure within a season or many; as a matter of fact, in good soils, performance has been admirable, but with increasing traffic, the problem has been an ever present one since the inception of bituminous surface treating in this state and one which we are rapidly correcting.

When such a project is designated for improvement, the initial step in procedure is to scarify the oil mat and bring the material into a windrow in the center of the road. It is then worked with a blade and roller or other applicable equipment until all of the chunks are reduced to the maximum size of the metal aggregate. The method we have found most effective is a pull grader propelled with a caterpillar tractor. The track grousers on the tractor supplemented by the units' weight supply an effective pulverizer.

Unless, in construction, the original mat was seriously over-oiled, additional bituminous material is required; about half of the pre-estimated amount is applied to the mat material in the initial processing, the oil, acting as a solvent, renders the material easier reduced. Following the preparation, the mat material is bladed into a windrow as close to one shoulder as practicable. Next by means of a blade, the untrenched portion of the grade is cut out and bladed off onto the inslope. The bottom of the trench is maintained at a slightly lower elevation than the original trench and inclined toward the shoulder more definitely than standard crown rates, thus to provide all opportunity for subgrade drainage and obviate possible moisture collection. It is, of course, imperative that the new trench overlap the original in order that the two will embody continuity of crown. To insure this, it is necessary to cut into the side of the original base course from two to six inches. The new trench and the widened portion of the road, created by the trench excavation, is watered, rolled, and bladed, until thoroughly compacted and a true section has been obtained. The newly prepared portion of the subgrade is covered with subbase gravel to the desired depth. Often conditions demand that additional material be added to the base course already in place augmenting the strength of the roadway section throughout.

Necessarily, the section being improved must be worked for half or two thirds width so that all mat material may be conserved and the operation requires delicate manipulation for preclusion of oversize from the base gravel with the mat material. Normally, the base course gravel used is pit run and in most areas the pits are such that a selection of material within easy range of standard specifications can be obtained. Our specification for pit run gravel is 3½″ maximum containing from forty to sixty-five per cent retained passing the four mesh. Selectivity is the prevailing rule to keep the fines sandy and clean and in the higher percentage brackets for density of the completed base structure. Much of our soil is gumbo—the essence of support when dry, but to which moisture is readily attracted and when saturated, forms a gluey substance easily injected into subbase voids by the pumping action of traffic. A dense and impervious base is of exceptional importance on a subgrade of this soil. Consequently, we exercise great care in keeping the base gravel free of material having affinity for moisture. Occasionally, the character of the subbase gravel compels sparing use of pretested binder or void filler in the achievement of a firm and dense surface to the base.

The most expedient means of placing the subbase gravel is by dumping it from trucks in thin lifts overlapping or shingling the spreads, a procedure obviating the necessity of processing the entire depth for proper gradation. Spreading gravel, watering, rolling, and blading, are concurrently carried on to effect the highest degree of compaction and provide a riding surface most convenient to traffic under construction. Seldom is it ever possible to employ detours, therefore, traffic must be carried throughout the continuation of the work. Completing one side, the windrow of oil mat material is bladed across the road to the completed side and the process repeated on the unimproved half or one third.

For equitable production, the improvement is usually prosecuted in three mile sections, tearing up the oil mat, base surfacing, and oil mat finishing, proceeding in proper sequence and succession. It is to be noted that in addition to a redesigned and reinforced base, extra roadway width from four to ten feet is also derived from this operation. This type of betterment, with additional quantities of from 1,500 to 2,000 cubic yards base course per mile and including all phases of the renovation will average from $1,200 to $2,000 per mile in cost. The finished product usually embraces a 1½″ thick 22′ to 24′ width bituminous mat, a subgrade width of 26′ to 30′ entirely covered with an eight to nine inch compacted gravel subbase course. A road of this standard delivers excellent service to 600 to 1,000 cars per day.

Advantageous equipment organization comprises one cat and blade with scarifier, five or six motor graders with scarifiers, and watering and rolling units. The cat and blade are employed for scarifying the oil mat in preliminary operations, two motor patrols for blading the gravel as it is placed, and two or three motor patrols for finishing and reprocessing the oil mat. Gang-wheeled pneumatic-tired rollers are used on subbase compaction and in conjunction with the conventional self-propelled steel-wheeled roller on compaction of the oil mat. Naturally, the amount of road equipment is controlled by gravel production. The equipment assembly described is that required for gravel production averaging from 750 to 1,000 cubic yards per eight hours.

On later projects unattended by the semi-trench where traffic has demanded more adequate bases, a process of base repair similar to the one heretofore described is used except that the disagreeable old trench is not present for contention. Happily, not all of our base repairs are of such extensive intent.

Increasing traffic problems and improved construction methods of later years are prompting and providing bases of twelve to eighteen inches depth designed and built for permanency. Even then from an overlooked combination of soil and moisture on construction the occasional repair is required.

On isolated repairs of only a few feet expanse, drudging hand methods are resorted to. In these instances, the oil mat and base course are manually removed and the offending soil is excavated to a substantial depth below the bottom of the subbase. The hole is backfilled with specified base material, hand tamped in stages to the level of the oil mat, and allowed to remain several days in this condition for the benefit of traffic compaction. When the refilled material appears to be thoroughly stabilized, the excess is removed to the bottom of the oil mat and premixed bituminous material is filled into the void. These repairs are more delicate in nature and individual in problem dictating deviation from routine repair practice as the exigencies of each demand.

Recurring and chronic disturbances in separated and exaggerated cases are disposed of by subgrade drainage with either open joint tile, French drains or porous or perforated pipe. And, since moisture is the confounding spirit of the majority of base failures, it is indeed rare when proper subgrade drainage will not provide a permanent remedial measure.

Shoulder Maintenance

By

C. P. HUMPHREY
District Engineer
Idaho Department of Public Works

In our district of Northern Idaho, there have been three sections of road receive special bituminous treatment of shoulders. All three were on our main north-south route, the North and South Highway.

The first work was done in 1937 on the fills of a four mile section through a rolling farming country. The embankments were from 100% common material classified by soils engineers as A-4. The amount of surface water from rains and snow is large enough so that erosion is a problem. By applying a bituminous treatment to the shoulders and building small wooden troughs, we were able to check erosion on fill slopes and prevent surface water getting beneath the oil mat.

One factor in our favor was that all fills had guard posts on each shoulder. This we knew would tend to keep traffic from running on shoulders and consequently the bituminous treatment would not have to withstand traffic abrasion.

Troughs were made from 2 by 8 planks and installed just inside the guard posts with the top edge level with the shoulder. By means of a hand nozzle or ''snivey,'' SC-3 was applied to the shoulder between the edge of the oil mat and the guard posts, at the rate of 0.3 gallon per square yard. This was covered by a granite sand which prevented the penetration of all the oil into the shoulder material. The sand and oil formed a tough asphaltic film on the upper portion of the shoulder. Any surface water on the travelway ran into the troughs and was carried to the bottom of the fill.

This treatment has been in place two winters with satisfactory results. There have been no noticeable edge failures due to water seeping into the subgrade, nor have the slopes been eroded. The fill slopes were seeded by the Soil Conservation Service and now the growth is heavy enough to permit removal of troughs.

The second section that received a bituminous treatment of shoulders was on a nine mile grade called the ''Lewiston Hill.'' The problem had been in the past that the edges on the ''hill-side'' of the oil mat were continually breaking. The mat was 18 feet wide and the ditch in most places is one foot wider. In the spring and fall, during the heavy rains, the ditch being small allowed water to seep beneath the mat. Heavy buses and trucks with their dual wheels placed a load on the edge that would cause the break-up when base was in this weakened state.

In 1938, small portions were reconstructed. Some additional width was obtained, allowing a little wider travelway. During surfacing and prior to oiling, a little variation from our standard was used. The subbase and base were crowned five inches. The oil mat was laid as a 5-2-5 mat with no trenched edge. Thus the crown of the travelway was decreased and the edge was higher above the bottom of the ditch. The mat was compacted well with special attention to the edges. Before the shoulder material was placed, a three foot strip was sealed with RC-4 at the rate of 0.3 gallons per square yard. The strip was placed so that 1½ feet on the edge of the travelway, the vertical base of the mat, and 1½ feet of the base were sealed. This treatment tended to stabilize the untraveled edge of the oil mat and to prevent water in the ditch from running beneath the oil mat. This section has been in use through two rainy seasons and one winter. It has proven quite satisfactory.

The third section was in a more arid location in the Salmon River Canyon. The base and mat are of a sand-gravel type with a predominance of sand. Some treatment was thought advisable to keep the shoulder material in place under the swiftly moving traffic. The mat being only 18 feet wide caused traffic to travel near the shoulder. The air currents kept whipping the sandy material out of place, leaving the edge of the oil mat unsupported. To remedy this condition, a three foot strip was treated with RC-3, 1½ feet on the mat and 1½ feet on the shoulder. 0.3 gallon per square yard was applied and covered with sand to prevent traffic picking up oil. This treatment has been in use for one year. It has checked shoulder erosion and sealed out surface water.

BASE STABILIZATION

There has been only one definite attempt at base stabilization made in our district of Northern Idaho. This consisted of four special experimental sections totaling 5,100 feet in length sandwiched in a nine mile contract construction project, built in 1936.

The location was on the Moscow-Bovill Highway just east of Moscow. The soil type is A-4, 5, the typography is rolling farm country, and the geology is eolian or windblown formation with practically no parent rock exposed in this section. The U. S. Bureau of Agriculture states that this soil type varies from 5 to 60 feet in depth.

This section was chosen first because it is in an area where base stabilization is needed to supplant the shortage of mineral aggregate and secondly because it was near the materials testing laboratory permitting laboratory control of proportioning, moisture contents, compaction, etc.

The design of the sections was as follows: (Above subgrade of native soil). Sec. A: 2600 feet long, 6 inch stabilized course of disintegrated granite and emulsified asphalt covered with 5 inches of crushed basalt surfacing.

Sec. B: 1000 feet long. A four inch course of disintegrated granite placed to break capillary rise of subsurface water. This was covered with 12 inches of the native A-4, 5 soil and the wearing course was 5 inches of crushed basalt.

Sec. C: 500 feet long, 5 inches of crushed basalt surfacing.

Sec. D: 1000 feet long. 2 inch layer of stabilized granite and emulsified asphalt covered with 5 inches of crushed basalt.

The general design of the remainder of the project was 6″ of 2″ maximum crushed basalt base course and 4″ of 1″ maximum crushed basalt wearing course.

All available data covering construction were published in a report dated November 1936. Since that time there have been six separate studies made of these sections. The information has been collected, summarized, and will be published by September 1.

These studies have been inspections of the travelway as to its stability and serviceability under traffic, repeated moisture determinations at different parts of each section, and a progress report has been kept in an attempt to determine the effectiveness of each design.

WINTER MAINTENANCE

Our district covers an area that has several types of winter. Each type has developed a different maintenance method of patching holes in oil mats:

1. Sub-zero temperatures with heavy snow: It is practically impossible to use bituminous premix or dry material with asphaltic penetration. Bins, sheds or some other method is used to keep crushed aggregate fairly dry, unfrozen and accessible. Mineral binder or soil binder is added. This is placed in hole and compacted. It may require a large percentage of binder but it is workable. It may be necessary to repatch during the winter but in the spring the hole will be patched with a bituminous material.
2. Little or no snow, some rain, cold: The wet season is short; the precipitation low. In these sections either heated premix, dry aggregate penetrated with heated asphalt or dry aggregate penetrated with emulsified asphalt is used. In either case, the material is stored where it can be kept as dry as possible. Before placing, the premix is heated usually on a piece of boiler plate over a firepit and tamped with a hot tamper. When using the penetration method, the asphalt is heated in kettles and applied to the dry aggregate previously tamped into the hole. If emulsified asphalt is used, no heating is required. Care must be taken to get hole rid of any free moisture. Experience is needed to determine the amount of asphalt to apply or the patches will bleed and become soft in the spring when hot weather comes.
3. Wet and cold with little snow: There are short periods when asphaltic materials may be used if the hole can be dried out well and the patch is allowed a short time to cure. Otherwise the rain will wash the emulsion from the aggregate or will penetrate the premix patch and freezing will cause disintegration. Usually a coarse sand or crushed aggregate is well mixed with a good binder and tamped into place. This will give fair service to the traffic but may need repatching during the winter. In the spring this will be replaced with a bituminous premix patch.

Repair of Concrete Surfaces in Utah

By

LEVI MUIR
Materials Engineer
Utah State Road Commission

and

WM. F. HUGHES
Soils Investigator
Utah State Road Commission

WM. F. HUGHES

In order to correctly determine whether to patch, seal, remix, or reconstruct a section of bituminous road, it is necessary that certain fundamental observations be made. These observations should be made by a trained man.

A report of the observations should include: (1) A soil survey; this survey to include (a) probable cause, extent, and type of failure, (b) depth and condition of existing gravel subbase, (c) condition of drainage structures, (d) position of water table at all seasons of the year, (e) soil conditions of subgrade.

(2) A survey of the alignment with particular attention being paid to present day and future traffic requirements.

(3) A study of the probable salvage value of the present road.

(4) A study of the financial condition of the state; whether or not, at the present time, the state is financially able to reconstruct.

After making these studies a proper decision, whether to patch, seal, remix, or reconstruct, can be made.

Because of the large amount of hand work involved in making patches, it is often economical to make very extensive repairs. In many cases where extensive repairs are necessary, a less costly method of making such repairs can be used with the thought in mind of holding only the present surfacing until such time as major changes can be made.

RESURFACING WITH NATIVE ROCK ASPHALT

In Utah we have encountered the same difficulties as other states in keeping Portland Cement concrete surfaces in serviceable condition. This problem has been solved quite successfully by resurfacing with native rock asphalt. For the best results certain fundamental rules must be followed. The rock asphalt must be ground fine enough so that all passes a screen having ½-inch square openings, and the minimum bitumen content should not be lower than ten per cent.

Our specifications call for a sandstone rock asphalt and require it to be supplied from one source and quarry for any one particular project. In obtaining the proper percentage of bitumen, it is often necessary to blend the rock containing the varying percentages of bitumen. This blending is done by crushing the material and then mixing in a pug mill without the application of heat.

In no case should the rock asphalt be laid when the air temperature is below 70 degrees F., or when the rock asphalt contains more than five per cent moisture, or when weather conditions are generally unfavorable. Best results are obtained when the weather is very warm, so that the rock asphalt does not have a chance to cool off during the night.

In laying rock asphalt on an old concrete surface, it is necessary that old bituminous patches and all excess expansion joint material be removed. It is also important that all bad sections of existing pavement be repaired; either by removing the concrete, or by jacking up the present pavement, or by some other method to render a fairly smooth riding surface. The surface should then be carefully swept of all foreign material and dust, and a tack coat of RC-4 applied at the rate of ten-one-hundreths (10/100) to fifteen-one-hundreths (15/100) gallon per square yard.

In order to produce a smooth riding surface, it is necessary to remove all irregularities from the old surface by means of a leveling course. One-half inch thickness is usually required to produce a leveling surface of satisfactory smoothness to receive the main course of rock asphalt, which is usually one and one-half inches in thickness. The leveling course is best applied by means of drag spreaders, arranged so that the surfacing material is deposited in the low points on the concrete surface and no material is placed on the high points. In some extreme cases it is necessary to place the leveling course in more than one application. Rolling is probably best accomplished by means of a roller of the tandem type, weighing not more than eight tons.

Before the surfacing course is placed, it is necessary to set side forms of wood or steel to the exact line and grade of the finished surface. Spreading of the surface course can be done by use of concrete type finishers, spreaders of the Adnun or Jaeger type, or by means of drags. All methods have proved satisfactory. The drag spreaders, developed by members of the State Road forces connected with this type of construction, have proved especially useful. They are at least twenty-two feet long and have four cross members, which are placed high enough to provide the necessary thickness of surface after compaction. The drag is fastened behind the truck; loaded with rock asphalt; the load is hoisted; and the surfacing material dumped in front of the drag. As the truck moves forward it pulls the drag through the rock asphalt, thus spreading it uniformly upon the surface.

After the rock asphalt is spread, its surface is lightly rolled with a tandem roller weighing not more than 8 tons. Rolling is a very particular operation. It must be done at exactly the right time, and during the entire period of compaction the surface must be kept closed to traffic. After the first rolling, further compaction is best done in the cool of the early morning. It has been found that if this preliminary rolling is done during the heat of the day the surface tends to push causing irregularities to develop. As the water evaporates, consolidation gradually takes place with further rolling. Final rolling is best done in the heat of the day with pneumatic-tired rollers, and is continued until the surface is entirely sealed.

The finished surface should be true to line, grade, and cross-section, and free from ruts, bumps, or other irregularities. If the required smoothness has not been obtained after the rolling, the low spots can be sprinkled with fine rock asphalt screenings and again rolled. It is important that the surface course be placed as continuous as possible, and in rolling, the roller should pass over the unprotected end of the freshly laid material only when the laying is to be discontinued for any length of time.

Many projects have been laid in Utah using this method and we find that the average cost is about $0.65 per square yard.

RESURFACING BY ROAD MIX

Some work has also been done using a road mix for resurfacing old concrete pavements. An open graded type aggregate is used and is mixed directly upon the concrete surface.

In this method the old concrete surface is prepared in the same manner as for the rock asphalt surfacing; after which a tack coat of 0.10 gallon of RC-4 per square yard is applied. After the tack coat has dried, a layer of rock chips, three-fourths inch in depth and averaging 35 to 40 pounds per square yard, is placed on the surface; and three-eighths gallon per square yard of RC-4 is applied to the chips, thoroughly mixed with blades, laid, and lightly rolled. It is essential that the aggregate used be entirely crushed rock, and range in size from ¾ inch to ⅜ inch.

After the mix has been laid and rolled, a light application (10 pounds per square yard) of key rock, ⅜ inch to ⅛ inch in size and preferably washed to remove all dirt and fines, is spread evenly over the surface with a broom drag, and is finally, lightly rolled.

At this stage of the construction, traffic may use the road if necessary before final application of the seal. Before making the final seal the surface should be swept clean. The seal is an application of ⅜ gallon per square yard of RC-4 and 25 to 40 pounds per square yard of stone chips, ⅜ inch to ⅛ inch in size. The finished surface is then rolled using either a pneumatic-tired, or tandem roller.

The cost of this type of surface treatment is about $0.30 per square yard, and has proved satisfactory on several projects in Utah.

The Repair of Bituminous Surfaces

By

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Maintenance Engineer
State of Washington

It is a fact well known to maintenance engineers that every failure of an oiled surface is due to a definite cause, and that before repair can be properly made, the cause must be accurately determined, and the faulty condition removed.

Failures are principally due to lack of proper drainage and insufficiency of base, and it has been well demonstrated that in order to produce a satisfactory and lasting oiled surface, the first consideration is a foundation for same. This foundation must be such as will successfully drain off waters that find their way underneath the oiled mat, and of a character of material, and depth of construction, as to resist any capillary action such as results principally from the pressure of clay in base or sub-base.

Here in the State of Washington practically all of our new construction of oiled surfaces is on stabilized base, which in the building up thereof, we are singularly fortunate in having in most localities, ample supply of natural rock of suitable size and quality, to form the sub-base or ballast course, and without necessity in a majority of cases, for any crushing operation, though it is sometimes desirable to screen out excessive oversize.

Not all of our oiled highways, however, by a quite large percentage, are of the ideal type above described, so as a consequence we have an abundance of failures, the principal of which are Scaling of Seal Coats, Potholing, Edge Failure, Alligator Surface, Rolling and Shoving. In addition to these types of failure which can occur in practically any season, we have in the early spring a considerable amount of Frost Boils, and also Break-ups due to overloaded trucks operating over roads that are soft due to the presence of moisture underneath the oil mat.

In general, the repair of bituminous surfaces is something that produces for the Maintenance Engineer, more ‘‘headaches’’ than perhaps any other type of maintenance. It is quite disconcerting, and very conducive to a display of bad temper, to start out on a tour of inspection and find on a particular stretch of your pet oiled surface, that since your last trip over, the patching crew has been over same, and left its appearance very much resembling one of Grandmother's crazy patchwork quilts. Patches of every shape imaginable, and of every hue under the sun; also the riding qualities of these patches leaving much to be desired.

There is little excuse for any of those ills just described, and one of the first things that should be impressed on the repair organization is, that patching can be done in such a way as to not create an eyesore. When the decision is made to patch a particular area, it should be sized up with view to completing on the surface at least, a rectangular area, and not try to follow the outline of broken surface. If the patch is to be a deep one, it can be dug out to the required depth for underneath reinforcing as necessary, and on the surface only a thin layer removed, covering just the sufficient area necessary to accomplish squaring up that is desirable. When it comes to actually apply the patch, careful consideration should be given to the quality, color and sizes of mineral aggregate to be used. The same careful consideration should follow to the type of oil or bitumen that is used. The mineral aggregate used should correspond as nearly as possible with what was used in original construction, and if like consideration follows to the type and grade of oil, the patch will in the course of just a short time blend more or less perfectly into the original surface that surrounds it. The riding qualities of the patch can be quite nicely regulated, particularly in cases where pre-mix material is used, by seeing to it that too large aggregate is not used in the main body of the patch, and by using pre-mix composed of smaller rock for ''feathering off'' on the edges.

One of the most common types of failure is that resulting from pot holes. These usually commence with a quite small indentation located in a wheeltrack. The continuous pounding by the wheels of vehicles, particularly heavily-loaded trucks, soon accomplishes from a small start, the making of a goodly sized hole. If, as quite often happens, the hole has not been plugged up and sealed in advance of rains or other precipitation, water gets under the mat and a sizable patch and undermat reinforcement becomes necessary.

Here in the State of Washington our maintenance of highways is organized to employ a system of long patrols. The lengths of these patrols vary according to local conditions, from a minimum of around 40 miles to a maximum of around 65 to 75 miles. The patrolman is furnished with a light panel body truck and this equipped with such tools as are necessary for light repairs of every nature likely to be encountered, such as broken guard rails, clogged up drainage, damaged or dirty signs, gravel on pavements at intersecting gravel roads, fallen trees across or along the travelled way, and the multitude of other little things that are of daily occurrence on most highways.

One of the important duties that are insisted upon as referring to these patrolmen, is that they carry in their patrol car, a small boxful of bituminous premix. Whenever they encounter what appears to be the commencement of a pot hole, they without further delay dig out as necessary and tamp in some of the pre-mix they carry. With this done there has been saved in a large majority of cases, the expense of repairing a much larger break by the organized repair crew which probably would not be along that way for some little time. This method of carrying on has been quite conclusively demonstrated to be a very effective preventative of more or less major break-ups resultant from an insignificant start.

It has also been found to be a good practice to seal all deep patches right away after installing; this regardless of whether the original surface has or has not been sealed. Even a non-sealed surface after being laid for a more or less considerable time becomes quite impervious to moisture. Any ordinary patch, regardless of methods of compaction, retains for some time a condition of porosity that invites the reception of moisture which tends to soften up and cause failure or partial failure of the patch. The application of seal is quickly and effectively accomplished at relatively small cost.

In this State we are running more and more largely to the use of pre-mix material for the repair of broken surfaces. It is becoming the common practice to mix up and stockpile quite large quantities of pre-mix; generally a sufficient quantity estimated to be enough to carry over for a year. All of our pre-mix is manufactured using for binder a mixture of 50% MC-2 and 50% SC-2. Some of this is stored in sheds and some outside in the open, but all is preserved in a sufficiently soft and workable condition as to be readily handled, and with no apparent loss of effectiveness. In the case of outside stockpiles it has been found desirable and very effective to seal the outer surface of the pile using for this purpose ordinary pipe dip. This practice definitely insures that no deterioration will occur in even so long a period of storage in stockpile as a year.

Maintenance of Bituminous Surfaces

By

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District Engineer
Michigan Highway Department

It is the purpose of this paper to discuss surface repair, seal and surface treatment of bituminous surfaces and surface treating old concrete pavements. It has been requested that the man actually doing the work be kept in mind in this discussion. It would therefore seem logical that the instructions, given the field men in the Maintenance Division, be made the subject matter of this paper.

When an asphalt or concrete surface begins to disintegrate, that is, when holes or depressions begin to appear in the surface, they must be repaired at once in order to prevent them from becoming larger and larger.

In the Michigan State Highway Department, the instructions to the maintenance man for the repair of holes from ⅜″ to 3″ deep are as follows:

Materials:

1. Tar T-1 for prime coat.
2. Bituminous patching mixture.

Procedure:

1. The area to be treated should be cleaned thoroughly by brushing (in some cases chipping) and if the hole is not thoroughly dry it should be dried by means of a gentle flame with a torch. This patching material feathers out very nicely, and it is not necessary to obtain butt joints at the edges of the patch.
2. Apply prime coat "T-1" by pouring a light application on to the area to be treated. Broom the material with a three tufted roofer's broom and thoroughly coat the bottom and edges of the hole. It is well to prime an inch or two beyond the boundary of the patch. The prime material should be applied as thinly as possible in order to avoid its coming up through the patch and causing smear-

ing later on. If necessary, in cool weather, the prime coat may be flattened out by applying a gentle flame with a torch.

3. The next step is to apply the bituminous patching mixture. In cases where the hole is over one inch deep the material should be applied in layers of one inch or less, with successive tamping between layers. Each layer should be leveled off with a lute or rake. It must be remembered that in deeper patches a greater thickness must be allowed for ultimate compression. Therefore patches should be made with a slight turtle back hump where the depth varies from the edge to the center. For example, where a patch is two inches deep in the center, it should be built up about ½ inch high in the center, and in the course of a couple of days with ordinary traffic the patch will be compressed flush with the grade of the pavement. As the depth of the patch feathers out, less additional material is required to allow for compression. In tamping, once over, only, will be necessary as the ultimate compression is obtained by traffic. Where traffic is sufficiently heavy to ditch or depress the patch, even though it was built high, additional material may be added subsequently by merely brooming the surface to roughen it and applying the additional material. No prime coat is required at the time of this subsequent application. Final check with a cord or straight edge is mandatory to insure the evenness of the completed surface.

Specifications for ''T-1'' tar will be found on the pages following as well as specifications for bituminous patching mixture.

When the patching of an asphalt surface becomes so extensive that the cost per year is greater than one-third the cost of a surface treatment or if it has dried out so that it is cracked and open, a surface treatment or seal coat becomes necessary.

The Construction Division of the Michigan State Highway Department has set up, in tabular form, a variety of treatments for distributor application of bituminous materials. These will be listed below with the tabulation. The Maintenance Division has adopted these same treatments on their maintenance work since they cover all types of treatments needed and the tabulation helps the field man in choosing the proper treatment and gives him the steps or operations in the order that they should be performed together with the proper quantities and combination of material to use.

The tabulations on the following pages cover the following treatments:
Class A-1 — Double seal on gravel base.
Class A-2 — Double seal on stabilized gravel.
Class A-3 — Double seal on waterbound macadam.
Class A-4a— Single seal on bituminous retread.
Class A-4b— Double seal on bituminous retread.
Class A-5 — Single seal on oil aggregate.
Class A-6a— Single seal on sheet asphalt.
Class A-6b— Double seal on sheet asphalt.
Class A-7a— Single seal with bond coat on concrete or brick.
Class A-7b— Single seal on concrete or brick.

For the sake of explaining the use of the above tables, the double seal for a bituminous retread, will be used. The tabulation for this treatment will be found under 4-b.

Under ''Quantities,'' the first operation consists of the application of the bituminous material at the rate of .25 gallon per square yard. The second operation is the application of the 25 pounds per square yard of cover aggregate. The third operation is the application of the second surface coat at the rate of .25 gallon per square yard and the fourth operation is the application of the second cover aggregate. This part of the table gives the field man the sequence of the operations and the quantities to use.

Under ''Combination'' in the same table is found a variety of materials which may be used for the different operations. Assuming that combination ''A'' is decided upon because of the availability of materials in that group, the first operation would require the use of MC-3, the second operation 26-B, the third operation SOA (200-250 penetration range) and the fourth operation 31-A.

If it is preferred to use tar, combination ''B'' may be used, where the material for the first operation would be T-6, for the second operation 26-B, for the third operation T-9 and for the fourth 31-B. This part of the table gives the field man a choice of the proper combination of materials.

Following are specifications for distributor applications, A-3, which describes Materials, Equipment Requirements, Construction Methods, Maintenance of Traffic During Construction, etc., which will apply to the work to be done in connection with the above tables.

It should be noted that types of materials and quantities to be used, in accordance with the tables, should be substituted for those shown in the specifications. Also when a prime coat is not contemplated, as is the case in the sealing of a bituminous retread, described above, the prime material and the descriptions of construction methods for the prime coat should be deleted from the specifications.

The Michigan State Highway Department has surface treated a considerable mileage of old concrete pavements, the surface of which was quite badly disintegrated. The results have been quite satisfactory and the life of the pavement has been materially extended. Under table 7a, the operations, quantities and materials are given for a Single Seal with a Bond Coat and under 7b operations, quantities and materials are given for a Single Seal. The use of this table together with the specifications will assure desirable results.

With the aid of the tables and specifications the average maintenance superintendent, with his experience, should be able to do a satisfactory job of surface treating on asphalt or concrete pavement.

MICHIGAN STATE HIGHWAY DEPARTMENT
SPECIFICATIONS FOR TAR, T-1, AASHO

Intended Use: This material is intended for use as a bond coat on concrete for both bituminous premix and skin patching.

Requirements: The tar shall be homogeneous and shall be a product consisting entirely of materials derived from the high temperature carbonization of coal, refined to the specified consistency as necessary to conform to the requirements given below. All tests will be conducted on the sample as received and the results determined on a dry basis.

	Min.	Max.
Water, per cent		2.0
Specific Gravity, 25°/25° C.	1.08	
Specific Viscosity, Engler at 40° C.	5	8
Distillation Test on Water Free Material, Per Cent by Weight:		
Total to 170° C.		7
Total to 270° C.		35
Total to 300° C.		45
Softening Point of Distillation Residue (Ring and Ball), degrees C.		60
Total Bitumen, per cent by weight	88	

Field Note: The temperature for application is 60° to 125° F.

SPECIFICATIONS FOR BITUMINOUS PATCHING MIXTURE

Description: This patching mixture shall be composed of mineral aggregates and bituminous material combined by plant mix methods. It is intended for use in general maintenance pavement patching.

Where reference is made to the 1934 Standard Specifications it shall be understood to include the Michigan State Highway Department Standard Specifications for Road and Bridge Construction dated September 1, 1934 with Errata and Changes effective June 10, 1936.

Materials: The materials shall meet the requirements specified in Division 8 of the 1934 Standard Specifications or as otherwise specified herein.

(a) Bituminous Material.
Asphaltic Oil, SC-4A 8.05

(b) Mineral Aggregates.
Coarse Aggregate—The coarse aggregate shall be crushed stone or crushed boulders conforming to the following requirements:

Gradation:	
Passing ⅜″ (square) sieve	100%
Passing No. 4 sieve	70–90%
Passing No. 10 sieve	0–15%
Physical Properties:	
Stone, Deval Abrasion for stone	8% max.
Stone, Deval Abrasion for gravel	30% max.
Toughness	5 min.
Thin and elongated pieces	20% max.

Fine Aggregate, 3 BC 8.04
Mineral Filler, 3 MF 8.04

Manufacture of the Mixture:

(a) Equipment Requirements — The equipment shall conform to the requirements specified under Oil Aggregate Surface Course—Class F, 4.15.03 (a).

(b) Preparation of Bituminous Materials — As specified under Oil Aggregate Surface Course — Class F, 4.15.03 (e).

(c) Mixing Requirements — As specified under Oil Aggregate Surface Course - Class F, 4.15.03 (f).

(d) Weighing Mixtures — For the purpose of determining the amount of all bituminous mixture manufactured and actually furnished, an accurate record of the weights of all such mixture shall be kept. Each load shall be weighed on an approved platform truck scale furnished by the Contractor which shall have been inspected and tested for accuracy and capacity by an official sealer at the Contractor's expense.

(e) Composition of Mixture — The constituents shall be combined in such proportions as to produce a mixture conforming to the following composition limits, by weight:

Passing ⅜″ (square) sieve	100%
Passing ⅜″, retained on No. 10 sieve	40– 55%
Passing No. 10, retained on No. 200 sieve	33– 52%
Passing No. 200 sieve	3.5– 5.5%
Bitumen	4.5– 6.5%

The proportions shall be varied within the limits designated as directed by the Engineer.

(f) Testing and Inspection — The Contractor shall notify the Department sufficiently in advance of starting the manufacture of the mixture so that arrangements can be made for testing materials and inspection.

Method of Measurement: Bituminous Patching Mixture will be measured by weight in tons.

Basis of Payment: ''Bituminous Patching Mixture'' will be paid for at the contract unit price per ton, which price shall be payment in full for furnishing the prepared mixture and delivering to the specified locations.

SPECIFICATIONS FOR ASPHALT CEMENT, SOA

(175 - 225 Penetration)

This asphalt is intended for use in maintenance skin patching. The asphalt shall be homogeneous, free from water and shall not foam when heated to a temperature of 175° C. It shall conform to the following requirements:

	Min.	Max.
Specific Gravity 25°/25° C.	1.00	
Penetration at 25° C., 100 g., 5 sec.	175	225
Flash Point (Cleveland Open Cup), ° C.	175	
Loss on Heating at 163° C., 50 g., 5 hr., %		1
Penetration at 25° C., 100 g., 5 sec. of residue from Loss on Heating at 163° C. as compared with penetration of asphalt before heating, per cent	60	
Ductility at 25° C., cms.	100	
Solubility in Carbon Tetrachloride,%	99.5	
Oliensis Spot Test	Negative	

NOTES FOR PROPOSAL OR PLANS
ESTIMATED QUANTITIES AND SEQUENCE OF CONSTRUCTION

4.07 Non-Skid Surface Treatment
(Distributor Application)
Class A

1. Gravel Base — Double Seal.

Quantities:

1. Prime Coat	.25 G/syd.			
2. 1st Surface Coat		.25 G/syd.		
3. 1st Cover Agg.			30#/syd.	
4. 2nd Surface Coat		.25 G/syd.		
5. 2nd Cover Agg.				25#/syd.
Totals	.25 G/syd.	.5 G/syd.	30#/syd.	25#/syd.

Combinations:	A	B	C
1.	T-3	MC-1B	AE-3
2.	T-9	MC-5	AE-3A
3.	*26-B	26-B	26-B
4.	T-9	MC-5	AE-3A
5.	†31-B	31-B	31-B

2. Stab. Gravel — Double Seal

Quantities:

1. Prime	.15G/syd.			
2. 1st Surface Coat		.2G/syd.		
3. 1st Cover Agg.			22 #/syd.	
4. 2nd Surface Coat		.2G/syd.		
5. 2nd Cover Agg.				18#/syd.
Totals	.15G/syd.	.4 G/syd.	22 #/syd.	18#/syd.

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*Note: 26-B is new proposed designation for the old 25-A modified as used in specifications.
†31-B is new proposed designation for the old 29-A modified as used in specifications.

Combinations:	A	B	C	D
1.	MC-1B	T-3	AE-3	RC-1
2.	MC-5	T-9	AE-3A	RC-2
3.	26-B	26-B	26-B	26-B
4.	MC-5	MC-5	AE-3A	RC-2
5.	31-B	31-B	31-B	31-B

3. Water Bound Macadam — Double Seal.

Quantities:				
1. Prime	.25 G/syd.			
2. 1st Surface Coat		.25 G/syd.		
3. 1st Cover Agg.			30 #/syd.	
4. 2nd Surface Coat		.25 G/syd.		
5. 2nd Cover Agg.				25 #/syd.
Totals	.25 G/syd.	.5 G/syd.	30 #/syd.	25 #/syd.

Combinations:	A	B	C	D
1.	MC-1B	T-3	AE-3	RC-1
2.	MC-5	T-9	AE-3A	RC-2
3.	26-B	26-B	26-B	26-B
4.	MC-5	T-9	AE-3A	RC-2
5.	31-B	31-B	31-B	31-B

4a. Bituminous Retread — Single Seal.

Quantities:		
1. Surface Coat	.25 G/syd.	
2. Cover Agg.		25 #/syd.
Totals	.25 G/syd.	25 #/syd.

Combinations:	A	B	C	D	E
	T-6	SOA*	RC-2	AE-3A	MC-5
	31-B	31-A	31-B	31-B	31-B

4b. Bituminous Retread — Double Seal.

Quantities:			
1. 1st Surface Coat	.25 G/syd.		
2. 1st Cover Agg.		25 #/syd.	
3. 2nd Surface Coat	.25 G/syd.		
4. 2nd Cover Agg.			25 #/syd.
Totals	.5 G/syd.	25 #/syd.	25 #/syd.

Combinations:	A	B	C	D	E
1.	MC-3	T-6	RC-2	AE-3A	MC-5
2.	26-B	26-B	26-B	26-B	26-B
3.	SOA†	T-9	RC-4	AE-3A	MC-5
4.	31-A	31-B	31-B	31-B	31-B

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* SOA (200-250 penetration range).

†SOA (200-250 penetration range).

5. Oil Aggregate — Single Seal.

Quantities:

1. Surface Coat	.25 G/syd.	
2. Cover Aggregate		25 #/syd.
Totals	.25 G/syd.	25 #/syd.

Combinations:	A	B	C	D	E
1.	MC-5	AE-3A	RC-1	RC-2	T-9
2.	31-A	31-A	31-A	31-A	31-A

6a. Sheet Asphalt — Single Seal.

Quantities:

1. Surface Coat	.25 G/syd.	
2. Cover Aggregate		25 #/syd.
Totals	.25 G/syd.	25 #/syd.

Combinations:

	A	B	C	D	E	F
1.	SOA*	RC-2	RC-4	MC-5	T-12	T-9
2.	31-A	31-A	31-A	31-A	31-A	31-A

6b. Sheet Asphalt — Double Seal.

Quantities:

1. Bond Coat	.2 G/syd.			
2. 1st Cover Agg.		20 #/syd.		
3. Surface Coat			.3 G/syd.	
4. 2nd Cover Coat				30 #/syd.
Totals	.2 G/syd.	20 #/syd.	.3 G/syd.	30 #/syd.

Combinations:

	A	B	C	D	E	F	6
1.	MC-3	MC-5	RC-2	T-8	T-8	AE-3A	MC-3
2.	26-A	26-A	26-A	26-A	26-A	26-A	26-A
3.	MC-5	MC-5	RC-4	T-11	T-8	AE-3A	SOA†
4.	31-A	31-A	31-A	31-A	31-A	31-A	31-A

7a. Concrete or Brick — Single Seal with Bond Coat.

Quantities:

1. Bond	.1 G/syd.		
2. Surface Coat			
3. Cover Agg.		.25 G/syd.	25 #/syd.
Totals	.1 G/syd.	.25 G/syd.	25 #/syd.

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* SOA (200-250 penetration range).

†SOA (200-250 penetration range).

Combinations:	A	B	C	D	E
1.	AE-2	MC-1B	T-3	MC-1B	AE-1
2.	AE-3A	MC-5	T-9	RC-4	SOA*
3.	31-A	31-A	31-A	31-A	31-A

7b. Concrete or Brick — Single Seal.

Quantities:		
1. Surface Coat	.25 G/syd.	
2. Cover Agg.		25 #/syd.
Totals	.25 G/syd.	25 #/syd.

Combinations:	A	B	C	D	E
1.	MC-5	T-9	RC-2	RC-3	AE-3A
2.	31-A	31-A	31-A	31-A	31-A

Proposed MSHD 1939 Designations (A.A.S.H.O.)	Present MSHD 1934 Designations
T-3	TC-2
T-5	TM-1
T-6	TM-2
T-8	TH-1
T-9	TH-2
T-11	TP-1
T-12	TP-2
TCB-5	TCP-1
TCB-6	TCP-2

SUPPLEMENTAL SPECIFICATIONS FOR BITUMINOUS SURFACE TREATMENT

Class A-3

Description: This work shall consist of three applications of bituminous material, as provided, applied to the prepared gravel base and three coverings of coarse aggregate. The width of treatment, unless otherwise provided, shall be 20 feet.

Materials: The materials shall meet the requirements specified in Division 8 of the Standard Specifications, or are specified herein as follows:

(a) Coarse Aggregate for Cover Material.

25-A — (Modified) Now 26-B in new proposed designations.

Passing ½″ square sieve	100%
Passing ⅜″ square sieve	75–100%
Passing No. 4 sieve	15– 45%
Passing No. 10 sieve	0– 10%

25-A gravel shall have a minimum of 50 per cent crushed material retained on the ¼″ screen. For the purpose of this test, crushed material shall consist of particles having at least one fractured face. 25-A stone or boulders (crushed) shall have a toughness of 6 minimum.

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* SOA (200-250 penetration range).

29-A — Gravel (Modified) Now 31-B in new proposed designations.

The cover material shall meet the requirements of 29-A gravel, except that the grading shall conform to the following requirements:

Passing ⅜″ square sieve	100%
Passing No. 4 sieve	80– 95%
Passing No. 10 sieve	0– 25%

29-A gravel shall have a minimum of 40 per cent crushed material retained on the No. 10 sieve. For the purpose of this test, crushed material shall consist of particles having at least one fractured face.

31-A — Stone or Slag.

Grading requirement same as 29-A modified above.

(b) Bituminous Material.
Prime Coat — TC-2, MC-1B.
Surface Coat — TH-2, MC-5.

Equipment Requirements:

Pressure Distributors.

The pressure distributor must be approved by the Engineer and shall have a capacity of at least 800 gallons, pneumatic tires, a tachometer or other approved device to secure accurate control of amount distributed, adjustable spray bars and nozzles and shall have quick shut-off valves.

Chip Spreaders.

The chip spreader must be approved by the Engineer and shall be capable of spreading the cover material uniformly for a width of at least ten feet and adjustable to spread uniform layers from 10 lbs. to 30 lbs. per square yard. It shall be a type that can be attached quickly to the rear of trucks and be backed over the cover material being spread. Revolving plate type of chip spreaders will not be approved.

Drags.

An approved drag to level and properly distribute the cover material will be required. Such a drag may be made from one layer of chain link fencing at least 10′ wide and 10′ long, so constructed and hitched as to cover one half the road width when dragged over the surface.

Rollers.

One or more approved rollers will be required to enable each surface cover material course to be promptly rolled. Rollers shall be self powered and may be three wheel or tandem and shall weigh at least 5 tons and not more than 10 tons. Devices for cleaning and oiling or moistening the wheels shall be provided.

Miscellaneous Equipment.

Sufficient equipment for handling and hauling cover material aggregate shall be provided to insure prompt and continuous covering of bituminous material.

A power broom shall be provided.

All necessary hand tools, thermometers, tickets for checking materials, etc., must be provided.

Construction Methods:

Prime Coat and Cover Material.

After the gravel base has been prepared and approved by the Engineer and when it is dry, the prime coat shall be applied with an approved pressure distributor at the rate of approximately 0.2 gallon per square yard, as directed by the Engineer, to 11 feet of the base width on one side. Immediately following the distributor and not more than 1,000 feet behind, cover material 25-A (modified) shall be uniformly spread with an approved chip spreader at a rate of 10 lbs. to 12 lbs. per square yard, if so directed by the Engineer. Application of cover material shall be made by backing the truck over the prime material, so that the truck and spreader wheels will ride on spread cover material. If the cover material is uniformly spread, the Engineer will not require dragging or rolling this course. Cover material shall not contain excessive moisture when it comes in contact with bituminous material. The prime coat and cover material shall be applied to the other side of the gravel base, as specified above, after one or two surface coats have been completed on the first side for a distance of not over 2 miles, or as directed by the Engineer.

Surface Coat and Cover Material.

The first surface coat of the specified bituminous material shall be applied with an approved pressure distributor at the rate of approximately 0.2 gallon per square yard, as ordered by the Engineer. The application of surface coats shall be one-half the base width, or 10 feet. Immediately following the distributor and not more than 1,500 feet behind, cover material, 25-A (modified), shall be uniformly spread with an approved chip spreader at a rate of 15 lbs. to 25 lbs. per square yard as directed by the Engineer. Cover material for surface coats shall be applied by backing over bituminous material so that truck and spreader wheels will ride on spread cover material. The cover material shall not contain excessive moisture when it comes in contact with bituminous material. This shall be followed at once by dragging the cover material with an approved drag and rolling with an approved roller so as to properly key the aggregate before the bituminous material sets. Any deficiency or excess of cover material shall be corrected before the second application.

The second application of surface coat bituminous material shall be made the same day, unless otherwise directed by the Engineer. This application shall be made at the rate of approximately 0.2 gallon per square yard and shall be immediately followed with cover material 31-A or 29-A (modified), at the rate of 15 lbs. or 25 lbs, per square yard, as directed by the Engineer, dragged and rolled, as specified for the first coat.

The Engineer will advise the Contractor how much of each size of cover aggregate to order. Variations in the relative quantities of 25-A (modified) and 31-A or 29-A (modified) cover material aggregate, or relative quantities of bituminous materials, as specified in the proposal will not be considered as the basis for any claim or adjustment in price.

The double surface coat and cover material shall be applied to the other side of the primed base as specified above. Care shall be used to avoid excess of bituminous material at laps and joints. All fat spots must be removed and satisfactorily replaced. Areas deficient in bituminous material must be corrected by hand patching. Since a slight excess of cover material will be spread on the final application, some of this may be whipped off by traffic; and before acceptance, the Engineer may require this material to be swept back on the surface with a power broom, dragged and rolled if necessary.

Maintenance of Traffic During Construction: Traffic shall be maintained at all times by part width construction, as specified above. During the construction of the prime coat, traffic shall be kept on the unprimed side, except that traffic may use primed surface after cover material is applied to turn-out for passing vehicles. During the application of surface coats, traffic must be kept off until cover material is dragged and rolled. The sequence of operations shall be so handled that there will be the least inconvenience and hazard to traffic. The Contractor shall maintain sufficient flagmen day and night, barricades, flares, and signs marked ''Fresh Oil'' to properly protect and handle traffic to the satisfaction of the Engineer. If in certain situations, in the opinion of the Engineer, it is necessary to provide controlled one-way traffic, the Contractor shall furnish the necessary men and facilities. All expense of maintaining traffic shall be incidental to the construction of the project.

Protection of Structures: When bituminous material is being applied, all structures shall be protected to prevent their surfaces from being discolored.

Weather Limitations: No bituminous material shall be applied during rainy or threatening weather, or when moisture on the surface to be treated would prevent satisfactory bond, or when the air temperature in the shade is less than 50° F., except by the approval of the Engineer.

Method of Measurement: The bituminous material will be measured in gallons at 60° F., temperature for material applied cold, and at a temperature of 200° F., for materials which are heated for proper application.

Cover material will be measured by weight in tons. The Contractor shall provide means satisfactory to the Engineer for determining the weight.

Basis of Payment: "Bituminous Material Applied" will be paid for at the contract unit price per gallon, which price shall be payment in full for furnishing, hauling and applying bituminous material.

"Cover Material Applied" will be paid for at the contract unit price per ton, which price shall be payment in full for furnishing, hauling, and placing cover material, dragging, rolling and completing the surface.

SECTION V

Round Table Discussion

Round Table Discussion

Presiding

D. L. CHENEY, Bituminous Engineer
Montana Highway Department
Assistant General Chairman

Mr. Cheney: This morning's session will be devoted to the Round Table Discussion. As most of you know, Mr. J .W. Powers, Materials Engineer of the Arizona Highway Department, sometime ago accepted the chairmanship of this section and during the past months he has been assembling questions and material which he planned to use here this morning.

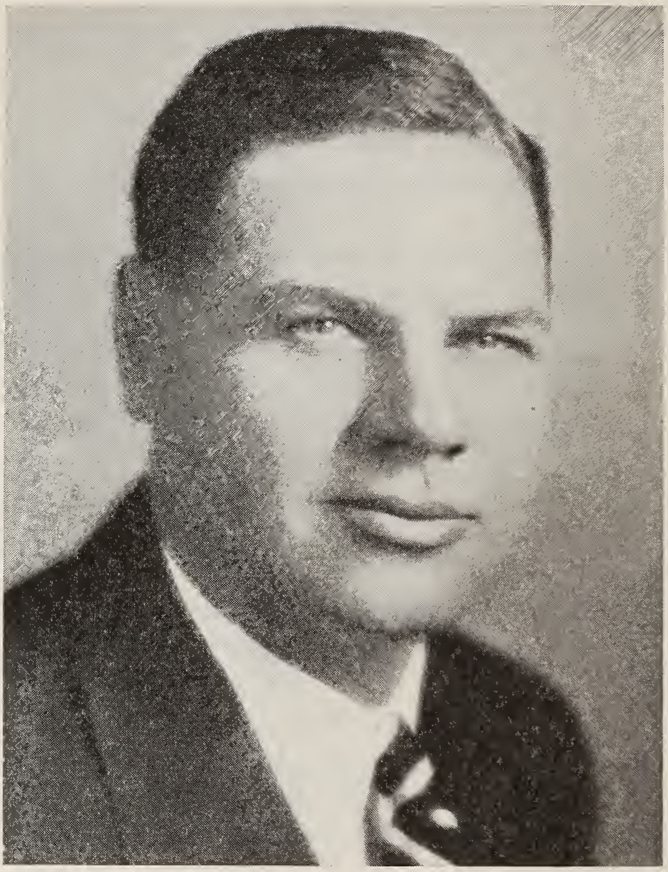
However, last Friday Mr. Powers telephoned me from Phoenix saying that they had had a terrific rain storm and flood there last week and their laboratory, being located in the basement of the Capitol building, was completely filled with water which came in through the windows, destroying practically all of their files and a large part of the equipment.

Mr. Powers said that due to the resulting confusion it would not be possible that he attend the Conference, and he asked me to express to the Conference his regret at not being able to be here.

I know Mr. Powers had done a lot of preparatory work on this section and had it well planned, and we certainly regret that he cannot be with us this morning.

However, we have been very fortunate in that Mr. H. G. Nevitt of the Socony-Vacuum Oil Company has kindly consented to substitute for Powers and lead the discussion. Mr. Nevitt will be under quite a handicap as he did not know until yesterday that he was going to have this job and Mr. Powers' notes and other material were all destroyed. So I hope you fellows will give him all the support you can and help him keep the discussion going.

I will turn the meeting over to Mr. Nevitt.



H. G. NEVITT, Manager, Asphalt Division
Socony-Vacuum Oil Company
White Eagle Division
Kansas City, Missouri

Mr. Nevitt: Mr. Chairman and gentlemen, I am pinch-hitting, as Mr. Cheney has said, and all I want to say to start off is that it is going to be highly informal. I am going to sit down, to add to the informality, and make a few brief remarks to start it off, and then we will get going.

These round table discussions have been a fine opportunity in previous years to catch up all the things that haven't been talked about. Sometimes some of the fellows have had a great deal to contribute. They have been rather reluctant to talk from the floor in the earlier meetings, but finding there was something they wanted to hear discussed that hadn't been taken up, they proceed to wake up, so to speak, and ask some questions which are quite pertinent. So it gives us a good chance to clean up a lot of things which haven't been discussed or perhaps haven't been thoroughly discussed, but the time seemed not to warrant it in some other session.

Just take the attitude that here is your final chance to get some of these things off your chest. As I say, it is going to be highly informal. I realize the wear and tear has been bad; in other words, depreciation is here, but I hope obsolescence hasn't set in yet. Allowing for that, I told Mr. Perry I wasn't even going to tell a funny story. As he expressed it, we are past that stage. We will just be informal and talk over anything we have to talk over.

If you get tired we will just quit. There is no reason to make it an endurance contest. And don't be afraid to talk. This is the kind of meeting that is really most valuable in lots of ways. We have the papers from this Conference, which are really wonderful, but you can read those over and study them later, when you can't discuss some of these things, either in the papers or out of them, like you can when you get in a little session like this. There is one advantage to the smaller sessions. It is just a little easier to talk back and forth.

With these preliminaries, I hope everybody will keep up front. When you want to leave, just get up and walk out. Don't worry about sitting in the last row so you won't disturb people, because they will be coming and going. Let's talk on the things we are interested in.

Various people were asked for questions, and I have a whole lot of questions, most of which are quite interesting. We have more questions now, than we have people to discuss them. On the other hand, I don't want to start off with these written questions if there are people here who have questions of interest to them, for the men who have asked these questions are here and we can just handle them from the floor. In other words, to fill out the discussion and follow a little logical procedure, I might resort to these, but I would rather just take questions from the floor. As I say, let's be informal.

In this Conference it has been emphasized that bituminous roads have been built from the base up, and we have talked about the soil underneath the subbase, the subbase, the base, and finally reached the bituminous mat. I feel a little bit that we have been sort of fatigued by the time we reached the top of the road. Since this is a bituminous conference, perhaps we ought to start the round table backwards by working from the top down. We have had a chance to build the thing, and now let's start to look it over, and start repairing it. We will hit the top first, so let's look at that.

I don't mean we have to follow any such procedure. It seems to me that perhaps there has been much discussion on the base, and perhaps a little less concern about the bituminous construction in which we are all vitally interested.

With those preliminaries, which I hope haven't been too long, the meeting is open for discussion. I would like to have questions from the floor first, rather than resort to these. I believe it is more or less in order, if you are agreeable to follow my suggestion, to talk about the design, construction or maintenance of the top course, the actual bituminous mat. Do I have anybody who wants to start the ball rolling?

R. C. Keeling: Kansas — I just wonder what the different states are doing when constructing mats on the bases, if they are shouldering up to the mats; say, for instance, dense aggregate mats 2½ and 3 inches thick. We have been constructing these mats 22 feet wide, and then dubbing them off along the edges. We have had a little trouble with pushing or shoving of the mat along the edge.

Mr. Nevitt: You say you are leveling them off? That is an excellent question, because I know that practice is changing, and I believe there is quite a conflict in practice at the present time. What do we have in the way of that? What is the practice? I think it would be fine to get what the states are doing at present and their ideas. How about you fellows from Missouri, to start off; what is the Missouri practice?

J. J. Corbett: Missouri — I can't furnish any really recent development along that line, because we haven't constructed any oil mat in the last couple years. As Mr. Keeling said, we more or less dubbed it off on the sides, and we had cases where it would push out. The tendency on recent work is to fold some shoulder material back up against the edges. Although we haven't built any mats, we have stabilized tops that we dub off similar to our past practice on mats. We bring back the material and try to shoulder flush with edges.

Mr. Nevitt: The material you shoulder up is granular, isn't it?

J. J. Corbett: Missouri — To a certain extent. In some cases it is just soil. We roll it with a sheep foot roller and try to compact it.

Mr. Nevitt: How about Iowa?

T. R. Perry: Iowa — We rather like to leave the mat on top with a drop-off, and then prime the shoulders. This cross-section is mighty fine from a drainage standpoint, but from a maintenance standpoint there are some rather distinct disadvantages. First, it is harder to roll the shoulders and fill ruts than on a uniform cross-section. Second, this section does handicap snow removal operations. I would not say it is the correct cross-section to use.

I was interested in some of the cross-sections we drove over on our trip out here. We noticed on some sections that the gravel extended from the edge of the mat to the outside edge of the shoulder. We took the liberty to dig into some of these roads and found that there was moisture underneath the gravel. We wondered at the time whether or not you were encountering any serious handicap or damage with this moisture working back under the mat. We found one or two mats where it appeared that the surface had been sealed clear across the base and shoulders before the mat was applied, and the loose gravel was placed above this oil membrane on the shoulders. I would be interested to know what conclusions you may have come to after using this cross-section.

Mr. Nevitt: Let's get your reaction to Mr. Perry's question in reporting on your state. I think it is best to take the round of states, and we can then argue it in general.

Mr. French, how about North Dakota? What is the practice there?

L. A. French:
North Dakota

We recently changed our specifications, and I believe now we are building up stabilized gravel bases from shoulder to shoulder. Last year we built our stabilized gravel bases and placed 1½ inch bituminous mats, the dense graded type, and on the shoulders we compacted gravel which was set up in the same manner that we set up our stabilized gravel, but the specifications for it were not quite as tight. We rolled and compacted those shoulders. Before putting the shoulders on, we shot the prime coat from shoulder to shoulder.

Mr. Nevitt: Your shoulder was flush over a prime coat, but not itself bituminous bound?

L. A. French:
North Dakota

That is right.

Mr. Nevitt: You think that will take care of the possible objections Mr. Perry brought up of keeping water out of the mat, and yet giving a flush shoulder?

L. A. French:
North Dakota

As to that, I don't know.

Mr. Nevitt: How about Wyoming?

C. F. Seifried:
Wyoming

I think Mr. Perry went through Wyoming from some of the remarks he made, and perhaps some of the other states, but really, they would apply to Wyoming. We have been in a transition state on this proposition. Many factors enter in.

Mr. Keeling mentioned the edge of the mat breaking off. We have come to the conclusion, rightly or wrongly, that the edge of the mat breaking off is not so much the failure of the edge of the mat itself, or that top layer, but is caused from possibly three—of course there are dozens of causes—principal causes. A lot of the roads are too narrow and, there is not the edge support to hold the load. When you put the slope right against the mat, common sense will tell you that a heavy truck wheel out at the edge will push out the subgrade.

Mr. Nevitt: Your top fails with the subgrade?

C. F. Seifried: Wyoming
Yes. This is caused by water getting in by capillarity, and getting under the mat. With a thoroughly compacted subgrade of good soil, I don't feel it gets in very far, but it does go under the edge.

The first point is to get some shoulder support. Widen your grades so there will be something there to hold it up.

As a second point, we felt that we didn't have enough gravel or metal under the edge.

Several years ago we had a great many roads that had just earth shoulders. With Federal Aid, we went over a great many of our roads and bladed off the shoulders in some cases up to a depth of a foot, depending on the soil. We shoved that off and put a solid shoulder of gravel on the edge, carrying our base clear out to the shoulder. Since that time—don't know whether that has been three or four years ago—we haven't had any earth shoulders on any of our roads.

The third point has been mentioned. We felt we should keep the moisture from going down into the shoulders so we primed clear across the base course. As a matter of fact, on a lot of jobs the engineers left the prime coat out of the center third and put more under the edges and on the shoulders. This moved the edge of the roof out so that capillary moisture had further to go to edge of mat. I am sorry it hasn't been discussed whether or not that prime coat is necessary. Whether necessary or not, we know some engineers have saved some of that prime coat. They have in all cases primed the shoulders and the edges.

We found, however, by digging down in the spring, especially after the snow had been on, the MC-1 film was useless because the moisture did get down through it. So as the next stage, we said, "If the MC doesn't hold it, let's see if the RC will hold it." We put the base course on, primed it clear across, and put on the mat.

In sealing the mat with the RC, or whatever was used—generally RC— we sealed clear out over the shoulder. So we had a film of MC, on top of which was a film of RC. Whether or not that film is shedding moisture off, we don't know as yet.

On new construction built this way, as far as the edges of the mat pushing out or as far as their breaking off, we have absolutely no trouble. We are building all our new construction on the heavier traveled roads with two 12-foot lanes. I have advocated two 12-foot lanes for years. They started out with 8 and 9. There were quite a few articles in the technical press that different states felt they should go to 11-foot lanes. Perhaps we have been extravagant. We have insisted on 12-foot lanes on our high-speed roads. We are constructing these roads to an 80-mile standard or 70-mile an hour standard, depending on whether you are considering safe speed or comfortable speed. It seems to me a man going down the road that fast needs a pretty wide place to shoot at, so we have stuck to the 12-foot lanes.

Going back to the shoulder on top of the MC film and RC film, we put a gravel shoulder on. I have thought about that considerably; we all have. Here we have a hard, solid mat in the center of the road, with no maintenance—they should be constructed so there will be no maintenance—but still we put the loose shoulder gravel out on the edge.

Now we have reached another stage in our development, and we feel that inasmuch as we are putting this prime clear out, putting the seal coat clear out, why not go another step and oil that mat clear out?

Of course, that brings in the question of visibility at night. We have a yellow stripe down the center, but the question is whether the edge of the road is marked well enough or not. I think I mentioned in the paper that instead of using just one yellow stripe in the center, it may be necessary to put a yellow stripe at the edge of the 12 feet. That is the stage we have reached.

I don't feel we are very extravagant. I think the maintenance saved on the shoulder will more than offset the extra cost of the oil in mixing that shoulder. The contractors are very well pleased with it. Under our old set-up insisted on their putting on the oil mat and sealing the road. Naturally, they have to seal it, and then they have to come along and put on the narrow gravel shoulder. We can't afford to put wide shoulders on the roads. It has to be a shoulder of from 2 to 4 feet. That is the present state of our thinking on designing the mats.

We are moving the edge out so far that we feel it isn't going to cause any trouble. We think we are going to get the waterproofing and the strength out there where we need it.

There is only one other feature of our design, and that has been mentioned by some of the other states. We all know that 80 per cent of our surface maintenance costs are edge and shoulder maintenance, so we skimp the center thickness of the base. We put a high crown in the subgrade. We have a comparatively small amount of base gravel in the center, but we have it thick on the edge.

Of course, we are talking about the newer jobs. We feel real smart about them and they all look pretty nice, but perhaps they won't do in five years. Maybe there will be something else.

Mr. Nevitt: Wyoming has apparently had a rather logical development in its design. I think the comment he makes on the shoulder prime is quite important, because it is a new thought which I am running into in several places, the idea of using your MC-1 to get your penetration, but due to its thinness—the fact that it does have to be thinned to get penetration and its high volatile content—you end up with a rather lean soil-asphalt mixture. So you follow that with an RC seal to seal the voids in this rather lean soil-asphalt mixture, and end up with a durable mat. In other words, it is a duplication of the same thought we have reached in mat practice, of a mix, which may be a little lean, protected from weathering by a seal coat.

How about Montana?

W. J. Leary:
Montana

Most of our failures on the shoulder edge have been on trench sections, on a mat that has probably been reworked and laid wider than the trench section, and has pushed out. On our later jobs, we have put what we call the subbase material from shoulder to shoulder, and on a late job have put a prime coat on the shoulder and then a 95 oil, with chips, clear to the shoulder line. That is our first job of that nature.

Mr. Nevitt: That was described yesterday. In other words, you have reached the point of carrying some sort of protection clear to the edge.

W. J. Leary:
Montana

Most of our mat shoulder failures have been due to not carrying the gravel under the courses out to the shoulder line.

D. L. Cheney:
Montana

Two years ago we carried out an extensive investigation of our failures on several projects. We, like Wyoming, knew that about 90 per cent of our failures were due to the base rather than the mat. After we tried to improve our base as far as our funds would permit, we still had failures. Out of that investigation, which we made on about twelve projects, we found that probably 90 per cent of the failures were base failures. Out of that 90 per cent, we found that about 25 to 35 per cent were failures which occurred through moisture from the shoulders working under the mat into the base. That was the reason for the oil treatment of the shoulders. We hope by this method to practically eliminate base weakening due to shoulder moisture.

Mr. Nevitt: Your logic is the same as that of Wyoming. Your proposed cure is slightly different, but the develpoment, so to speak, in your experience is the same.

I might speak for Oklahoma, where I had quite a talk with Jones, the maintenance engineer, on this subject last year. At that time, he was very strong for the same idea of sealing clear over to the shoulders, while just running the mat out to its seal width and leaving this little rise where it ends. He is doing something that nobody else has brought up. He puts an extra seal application right over the shoulder where this rise comes. He contends that by making that a little richer it tends to push out and pound down, not very much, but it stays in that condition. I haven't seen any of these jobs recently, but I know he felt quite satisfied to leave it that way. It does give it this extra little seal, about a foot or 2-foot spray when sealing to give a little extra oil right at the break of the mat down to the shoulder.

T. R. Perry:
Iowa

I am interested to know what grade of RC you used on top of the MC-1.

C. F. Seifried:
Wyoming

On the jobs we have built in the past it has been the RC-2. We have gone practically entirely to RC-4 on our newer construction.

Mr. Nevitt: You get the same result as Montana, but you don't plan to put any gravel over it?

C. F. Seifried:
Wyoming

No. With wide mat and flat slopes, wide shoulders are not needed.

I don't quite agree with Prof. Moyer's contention that there should be a surfaced or an oiled lane for parking or for emergency use. I feel that if we had an extra width of shoulder wide enough for a lane, it would be used as a lane just as in the slides he showed of California. Perhaps we have gone a little bit too wide, and should hold it down to two lanes, with plenty of shoulder support. On the prairie roads with the flat slopes, we feel there is no danger. Even a truck can pull down at the side and into the borrow pit without much trouble.

You mentioned Oklahoma shooting the edges of the mat. We have done that in Wyoming for several years, both on construction and maintenance. It has given us about as heavy a shot of RC on the edges as it will take. We have trouble, of course, with the edge not getting the traffic. It doesn't get the so called kneading action that we don't talk about any more. We really have to put enough asphalt out there to hold it. It doesn't get any traffic compaction, and it isn't worked by the traffic as the center of the road is. We have to use more RC and it holds by itself without being compacted. I think the edge seal is very important.

D. L. Cheney: Montana — I would like to add to what Mr. Seifried has said about the RC shoulder. We tried that form of treatment a couple years ago on several sections. We found though that it was more or less temporary because, during the winter and snow removal operations, trucks pulled out over the edge of the mat breaking through the treatment, and cars or wagons going out there broke the RC crust, and in one or two years on most of the jobs the shoulder seal was ruined. I would like to ask Mr. Seifried how long their treatment lasts, whether they plan to do it every year or two, or whether it is just temporary.

C. F. Seifried: Wyoming — I think we will probably keep it up. It is true that we do have some difficulty, not so much due to snow removal as to sheep herders and narrow steel tires. We talk about 500 or 800 pounds per tire width, but they come along with their sheep wagons with ¾ or 1 inch steel tires. We are getting more pounds per square inch from these so called light sheep wagons than we do from the heaviest trucks. We have had a heck of a time. The tires do go out there and smash down the RC seal and almost ruin it in many cases. Fortunately, that hasn't been general. In that case, of course, the RC seal is broken down. Those are the only places we have had trouble. I don't think we have had any particular trouble with the heavy trucks or snow removal equipment.

Mr. Nevitt: I think that is the old problem of making annual repairs on a light structure versus the fixed changes of a permanent structure.

T. R. Perry: Iowa — I think Mr. Seifried has sounded a warning which we probably should all give a little thought to, and that is the proper width of roadway. Some of our early mats were built 23 and 24 feet wide from shoulder to shoulder. As a safety proposition, that road is probably too narrow, but we did not have the failures on the outside edges of the mat that we have had since we started to build, 6, 8, or 10-foot shoulders. One extreme usually follows another, and maybe we were a little short in width on our early roads, but I am afraid we are too long on the last design. Perhaps there is a happy medium which will bear pretty close consideration.

J. C. Black: Illinois

On this matter of shoulder to shoulder surfacing, I remember Mr. Seifried spoke yesterday of a 32-foot pavement with no shoulder. That is a 3-lane pavement. I don't carry the accident record in my head but, as far as I can recall, the 3-lane pavement has a pretty bad record for accidents, even when equipped with shoulders. I realize there is a center line down the 30 or 32-foot surface, but nevertheless, you have the dimensions of a 3-lane pavement there.

I wonder what anyone else might think about it. It looks rather bad to me. I think the 3-lane pavement has a much worse accident record than the 2-lane. Possibly I am wrong on that; I am speaking from an indefinite recollection.

Mr. Nevitt:

I know that what Mr. Black says is true statistically. Mr. Black, I believe the statistical record on accidents on 3-lane pavements was in the East where they were deliberately built as 3-lane pavements, whereas these are 2-lane pavements, patrolled and marked and otherwise administered as 2-lane pavements and 2½ lane pavements. You can't sneak over enough into the other lane to get by them without hitting another car. I have seen some of that myself.

Interpolating my own opinion, I question that you can say these are 3-lane pavements. You might just ask these gentlemen if there is any tendency to operate them as three 8-foot lanes, as well as 12-foot.

C. F. Seifried: Wyoming

There certainly is. That is the warning Mr. Perry tried to bring out. I think we have gone too far in that matter. On some of our new construction that has just been opened to traffic, where we have not had time to put the center stripe down, some drivers are using them as 3-lane pavements.

I think it was Mr. Templeton who mentioned the other day that he was driving on one of these roads when a fellow passed him, and another fellow passed both of them, and the three cars were alongside. It was operated as a 3-lane pavement. That was not the intent.

The point I wanted to bring up was that in widening these shoulders and oiling them, that is the very thing we are apt to wind up with, whether we want it or not. We don't control the traffic. People drive as they please in Wyoming, and I think they do in other states. If we have a patrolman every half mile, maybe we could control those things, but we turn the people loose and say, ''Here it is,'' and we have done the best we can. We have to leave something up to them. We can't have patrolmen, signs and signals every few miles.

The point brought up here I think we are all agreed on, that we must be careful not to have our shoulder on 2-lane pavements so wide that they are used as 3-lane pavements. There are probably not more than 6 or 7 miles out of 3,000 or 4,000 in the state of Wyoming that requires 3 lanes. Let's get these lanes wide enough to give shoulder support and quit there, and not try to carry it to extremes, as Mr. Perry said, because whether we want it or not they will be used as 3-lane roads, and that is the one thing we don't want.

R. M. Schwegler: Oregon

I would like to cite an example of a job we constructed in Oregon in building a highway near Bonneville Dam. We have a 22-foot pavement, two 11-foot lanes, and we have about a 10-foot shoulder. We placed a surface treatment on this shoulder, trying to effect a better design, in line with Mr. Moyer's recommendation. This road carries between 3,500 and 4,000 cars a day. Traffic is using this section as a 4-lane highway and not as a 2-lane highway. We are very much surprised that our light surface treatment is carrying that traffic as we have a lot of trucking on the road. The Columbia River Highway adjacent to this reconstructed section is narrow and crooked and 25 to 30 cars will pile up in a line. When the reconstructed section is reached, the heavy trucks pull out on the shoulder and wave everybody by. The slow traffic also pulls out as faster traffic passes it and the road is driven as a 4-lane highway. As far as that shoulder being effective as a shoulder for people to park on, change tires, and so forth, it is absolutely a failure. In mountainous country, I don't think you can depend on a shoulder for that purpose. You have to provide well established parking areas if you wish to provide that safety. I am just citing that as an example of an oiled shoulder that people will use as a highway lane.

Mr. Nevitt:

I wondered, when I listened to Mr. Moyer and he showed the nice 8-foot shoulder, whether he would educate his traffic to keep on what is usually the passing lane on a highway, or whether they would do as Mr. Schwegler says, use it as a 4-lane highway, with the slow traffic staying over on the shoulder where they don't belong, according to the theory of the design. If you did get them educated to stay in the two center lanes, would some of the cowboy drivers start passing on the outside to make sure they got by. It looks as though we need to build up a little experience on these wider lanes before we can go too far. I think the precautions brought up seem to be well taken.

We have talked quite a while on this matter. I don't want to cut off discussion, however. Are there any other states that haven't been called on that have some comment to make?

Art Loforth:
Nevada

On the reconstruction of our old 24-foot roads, which were 24 feet from shoulder to shoulder we are coming to two 12-foot lanes plus 6 or 7-foot shoulders, usually 7, and the subgrades topped with a type 1 base, we call it, full width, and primed. We have been priming with SC-1A, but recently we have gone to MC-1, which I think may be a little more satisfactory. The main thing we have been harping on is an oil shoulder. As I say, we have the two 12-foot lanes, and in some cases an 8-foot shoulder, and we road mix those.

The common practice is, after the base has been primed, to dump your material on the shoulders. It usually takes about 7 or 8 tons to a station, and then you road mix it, line it in with the windrow, and after your two 12-foot lanes are laid then come along on your shoulders and blade the windrow in.

Mr. Nevitt: Is that flush with your mat, then?

Art Loforth:
Nevada

The same thickness.

Mr. Nevitt: It may be a different construction, a different design than the actual mat, but the appearance is a continuous surfacing across, shoulder to shoulder.

Art Loforth:
Nevada

It is. In the pavement section of it we use an SC-4-5 where we blade lay it, and we aim to blade lay where we can take care of travel on detours, but where we can't furnish detours we use an SC-6 and machine lay it. The type of material used on the shoulders has been SC-2, and recently SC-3, and the only reason for the change is a change in price. We have a very decent price on everything except SC-2. One of the independent companies isn't equipped to handle the SC-2, and the result is we get a very decent price on everything else.

I have no doubt there is a tendency to make an extra lane out of the 38 or 40-foot width, but so far with the center stripe, the travel is staying pretty well to the center and there seems to be a pretty well defined line of demarcation on the shoulders.

We seal the surface with an MC-4 emulsified, using screenings, half-inch screenings, and then on the shoulders we seal with an RC-2 or, in some cases, MC-2 without any cover. The result is that you have the 24-foot width in the middle with your light color, and the outside with the dark color. I think, for that reason, we are not entirely out of line.

Mr. Nevitt: It is interesting that he does get a change in the color from the shoulders to the mat by the chips on the mat proper, and the shoulders left black.

D. L. Cheney: Montana

We have had practically the same experience in that connection, but we have tried to go at it in a little different way. I think, as explained yesterday, we use in our seal 95 plus road oil and half inch chips, carrying the seal and chips across the mat and clear out to the edges of the shoulder, first of course, priming the shoulders with one third gallon of SC-2. That leaves a perfectly white surface completely across the entire roadway.

Following the sealing, we shoot the shoulders from the mat out to the roadway edge with about a quarter gallon RC-1 which leaves the shoulders very black and the driving section white. From our experience so far, I don't believe we will have a great deal of difficulty with traffic going out in the black section as in addition to the change in color we also have a drop-off from the mat to the shoulder section of about 1½ inches which is not dangerous but is quite noticeable.

Mr. Nevitt: If we are going to have the chips to separate the shoulder from the mat, we are going to have to make certain our chip construction doesn't result later in a black surface which is caused by bleeding.

Let's hear from some other states on this. Is there any further comment on it? I think we have carried this through in pretty nice fashion.

D. L. Cheney: Montana

While we are on that subject of the use of 95 plus in sealing, we know from our experience that some discoloration is going to occur. I heard over in the lobby last night that some of the southern states are using a special grade of asphalt in sealing and are using ¾-inch chips, and they contend they have no bleeding. If there is anyone here who has used that material, or knows anything about it, I would like to hear from him.

Mr. Nevitt: Can anyone here discuss this new asphalt?

As an asphalt man, I might say I feel certain that it is something we can take care of; that the oil companies will develop a product which will not bleed through, with proper care, proper chips, and so on. We are working on it ourselves, and I think we have some ideas. However, it is a little bit early for me to say anything about it.

Is there anybody from the southern states who can report any developments along that line?

R. C. Keeling: Kansas — One of the contractors from Kansas was down in the South and bid on some of the work in Mississippi, and I noticed in the bid proposal they used plenty of aggregate. It seems as though that is one thing they have been insisting upon.

Mr. Nevitt: I think a lot of agregate and a very quick setting up product which promptly holds what aggregate is going to hold, with the rest kicking off, so there is practically no working up or bleeding, is required.

I think as we follow these things through we get a fine picture of what the trend is from these detailed discussions. While we have spent quite a little time on that subject, speaking for myself, I feel we have learned pretty much about it.

I. E. Russell: Wyoming — Some of the states seem to be using MC-s for sealing purposes, and I would like to know if those states have also used RC-s and MC's in the past, and what their idea is regarding the best material.

Mr. Nevitt: I think that is a good subject, because there has been a lot of discussion on sealing in past meetings. There hasn't been much this year, but some of the discussion here and some of the questions outside indicate that there is a great divergence of practice on sealing.

Mr. Russell has started the ball rolling. Can we have some comments from those now using MC's, and their previous experience with other materials, RC's in particular? Do we have some comment on the proper seal material? There are certainly as many different ideas as there are states, so it might be a good subject to discuss.

I have submitted a question here which I think is quite good. There are a couple of questions, but essentially they amount to this: Is sealing a permanent measure or is sealing merely a temporary measure? That is, does the seal we put on permanently correct or tend to correct certain defects in the mat, and once they are corrected, not necessarily by the first seal but by a series of sealing, from then on are you practically through with sealing as long as the mat is in good shape, or is sealing merely a temporary measure which has to be renewed every two or three years, or indefinitely, as long as the mat maintains its present shape? In other words, the question of the permanency of sealing as a maintenance measure.

Another question, which was a little different, was to the effect: Does sealing, from the weathering standpoint—in other words, you might say the permanency of the material, rather than from the wear standpoint—last as long as the mat, or are you sealing with the idea that you will concentrate your weathering in the seal coat and when your seal coat is gone you either have to renew it or have to renew the road, so to speak, put a top course on it, or something?

Can we have any discussion on that?

Art Loforth: Nevada

I can think of two sections built in 1929, ten years ago. They were never sealed. The practice of sealing didn't commence until about 1932. Except for those two sections built in 1929, I would say the life of the other sections, which have been down eight, nine or ten years, has been the result of effective seals.

On seal coats applied without any covering, say, anywhere from one-eighth gallon down to one-tenth, we found the average road constructed with a fairly lean mix and immediately sealed with, say, one-eighth gallon and in all cases until the last two years with an MC. I believe their life depends on sealing.

With the weather conditions we have in Nevada, I would say we are required to re-seal about every four or five years.

On the question of whether it is permanent or temporary, I would say a seal coat is temporary.

Mr. Nevitt: That is true even though you put no cover on?

Art Loforth: Nevada

Yes.

Mr. Nevitt: Do you find that failure or need for re-sealing every four or five years is due to the wearing off of the surface layer, or due to the need of getting more asphalt down into the mat, as well as some on the surface?

Art Loforth: Nevada

I would say one-eighth or one-tenth gallon is a pretty thin sheet of asphalt. I think a lot of it comes from oxidation, carbonization, and some of the other words we have listened to in the last few days.

Mr. Nevitt: Your idea then is that the reason for replacing the entire seal material is partially due to wear perhaps, and partially due to its removal or neutralization, so to speak, in effectiveness by other agencies.

Art Loforth: Nevada

Yes. If we can wait a couple of weeks, or two or three months, we have noticed the original seal will penetrate very readily, and ordinarily we won't have to sand it. However, after three or four years the surface will appear to be very lean and about ready to ravel. When you seal then with one-eighth or one-tenth gallon, it is very noticeable that the seal doesn't penetrate, and in most cases we have to sand unless the travel is so light we can afford to keep it off until it does dry. We have seen sections sealed after four years with one-eighth gallon, and it didn't dry for four or five days—using an RC-2, also.

By the way, the first seal coat we applied was an SC-2. I think the following year we started in with the MC's, and continued the MC's althogether until about two years ago. Since then, we have split between MC-2 and RC-2. I am a little partial to RC-2 myself. It is a little more apt to penetrate and dry quicker.

Mr. Nevitt:

There are a couple of interesting points there. Apparently, Nevada has slowly worked out from the slow curing to a tendency to get the most rapid curing, which I believe is the tendency in other states.

He also brings out another point which I think is quite important, judging from discussions I have had in various parts of the country on this seal coating. As I see it, there are really two types of seal coating, or two things that are done. One is the penetration of the mat; in other words, getting the seal down into the mat, with practically a lack of need for cover at the start, provided you have a non-skid surface. The other is the situation which he later reaches, of having the voids in the mat practically filled, and the seal essentially a surface layer cementing chips or sand to the surface to give a new wearing surface.

I think much sealing is a combination of the two, where part of the seal material goes down into the mat and tends to fill the voids, and the remainder holds on the surface the sand or chips. Some is purely penetration, and some is purely a layer on top to hold chips, but the functions of the two things are a little bit different.

I would like to bring that out because there is confusion on this subject on when we have been sealing with a soft penetration asphalt, as is the case in some states. We know we get very little penetration so we are putting on a surface layer only. The other extreme is a seal without any cover coat at all and, at least at the start, with a deep penetration. These are two quite different operations with different results.

Is there some more discussion along that line?

Rex Whitton: Missouri

I might say that on some roads under maintenance in Missouri we have used both those methods you speak of on the same job. That is, we go through and hit the dry spots with penetrating MC or SC, and then follow up in a week or two with an RC or penetration asphalt.

You say there are many different methods of sealing in the different states. There are also different methods of sealing in Missouri. I would like to know which is the best method. Various districts or divisions in our state use SC, MC, RC, and penetration. We still wonder which is the best.

Mr. Nevitt:

I think it is fine we are bringing this out, because that is certainly true.

I just recently visited one of the ten districts in Minnesota. The engineer there had quite a maintenance problem in sealing, and in just his district alone he used MC-3, RC-1 special, RC-1, and one or two kinds of tar. I remember those; he may have used some more—yes, he used some SC somewhere down the line. I think he used the SC because he had part of a car handy.

This same situation seems to exist everywhere, that there are a lot of different kinds of materials being used, and nobody has quite decided in many cases what is the best. I really think part of the trouble is due to lack of this distinction between the two operations, the penetration and the surface layer. I am satisfied that was the case in Minnesota. Where he had effective sealing, he had the material which had just the right characteristics to give him both operations that Mr. Whitton has done in two stages. In other places he perhaps didn't have the combination or the one he wanted. I think Mr. Whitton's remarks are interesting, that he is attacking the problem in its two phases by first sealing the voids in the present mat, and then proceeding to put on his cover course, or wear course for non-skid and other characteristics.

Can we have some more discussion?

V. G. Gould: Iowa

I don't pretend to know the correct answer to sealing, but this is the impression I have at the present time and I offer it just as a suggestion: That seals less than one-quarter gallon, either with or without cover, be considered temporary; that seals of a quarter gallon, or greater, covered carefully with cover aggregate and rolled to the extent that all cover aggregates are fully imbedded, be considered permanent, in the sense that such work may be considered permanent, and that the type of bitumen used preferably of a medium viscosity.

Mr. Nevitt: That is a very definite statement of opinion, and quite interesting.

L. L. Marsh:
Kansas

It seems to me that the proposition of sealing the mat and the type of oil used depends a good deal on your past knowledge and experience in that section. Very few sections in Kansas are sealed because the mat is wearing off. It is because of checking, cracking, or something like that.

Three different types of oil are used in Kansas generally. One is an MC-3 or possibly 4, if it is in the summer. It depends a good deal on what we want to do, whether we want to penetrate the cracks or build up a greater asphalt surface. Then we go to RC if we do late fall sealing; I think it penetrates and sets quicker, and also will not bleed so readily next spring when it gets hot.

On some of our stabilized base work we have attempted to use an AP, which is a penetration asphalt which I believe is quite successful.

Mr. Nevitt: Mr. Marsh makes the point that we are engineers and ought to analyze our conditions and prescribe the material the diagnosis calls for, which I think is something we always must keep in mind. The conditions we are trying to overcome are not the same, and therefore we must perhaps have different cures. However, I do believe analysis of the sealing operation is needed, and quite evidently has begun, from the discussion. We are beginning to get some analysis of what we want to do with the seal coat, and we are beginning to get a prescription based on this analysis.

Clif Hallvik:
Idaho

I might recite our problem. When you set up an oil project the Bureau wants you to finish the darned thing right now. We don't like to seal the same year if we can get away from it, but the Bureau doesn't like to keep the projects open to the following year for sealing.

We have set in all contracts an alternate, either MC-4 or RC-4, leaving to the judgment of the engineer at the time of the sealing operation which one he prefers to use. In case his mat should be of a certain character, or because of weather conditions, he might prefer MC-4, or he might take an RC-4 at the same price.

Mr. Nevitt: Putting in my own oar, I really think some of these can be analyzed. Do we want rapid curing or do we want medium curing? What do we want? If we want it, we should use it. Personally, I wonder on what the judgment of the engineer is going to be based.

D. L. Cheney: Montana

In this discussion about seal coats, there is something that is quite confusing to me and, I think to a lot of us. We talk about sealing with cover or without cover, and mention types of bituminous material used, but we don't draw any line of distinction between those two different types; that is, where we seal with cover, and without cover. One man will get up and say, "We use a certain grade of asphalt in sealing," but he doesn't state what type sealing he does and why he uses that grade of asphalt for it.

In Montana it costs from $90 to $150 a mile to seal with straight bituminous material such as RC-1, RC-2, or MC-2 without cover, and it costs anywhere from $500 to $850 a mile to seal with cover. I think there is just as much difference between those two types of seal as there is between a dust palliative and a 2½ inch road mix mat. I think we should have a definite distinction between a seal with cover and a seal without cover. But to answer the question, I would say that in Montana our light seals without cover are temporary and heavy seals with cover, or armor coats, we consider permanent.

Mr. Nevitt:

We ought to define what we are trying to do and what we are talking about doing when we discuss sealing.

J. C. Black: Illinois

The village of Winnetka, Illinois, has pursued a system of continuous maintenance for somewhat over twenty years —I don't know just how long. They have a lot of old-narrow, water-bound macadam, and they have fixed them up gradually with treatments of various sorts. They seal, as I recall, every three to five years. I don't know what they started with, but lately they have been using tar. They use torpedo sand with it—I presume somewhere in the neighborhood of ¼ inch. The result of this long period of work has been that they have fixed up those streets. They are not perfect, not ideal surfaces, but for a residential community of some 12,000 or 13,000 people they are pretty close to ideal. They built up the sections next to the curb and so got away from some of the excessive crown. The surfaces keep in good shape, and I have never known of any water going through, or any trouble. I don't think there is very much wearing off of this seal. They put on the sand and it is built up gradually. I know in one place the total thickness above the old surface is around 1½ or 2 inches.

I don't know whether they used tar or asphalt in the earlier days, but they have been using tar lately and it has been working beautifully.

Mr. Nevitt: Mr. Black brings a third variation into the seal problem. We first have sealing to maintain the present surface, and sealing to build a non-skid surface but not essentially to build up the mat; then the third, a successive series of seal coat to build up and improve the quality of the mat. We are here changing the type of pavement, so to speak, yet all of them are described as seal coats. I agree with Mr. Cheney that we need some differentiation if we are going to talk about them intelligently.

R. M. Schwegler:
Oregon

We don't have a very definite definition or line of demarcation between an armor coat and a seal coat. By increasing the quantity of asphalt and the quantity of chips, you more or less gradually work into an armor coat. Has anyone a clear picture of where one stops and the other starts?

One point I would like to bring out is that you can get into a vicious circle in using heavy seals. If you maintain a definite non-skid surface of good appearance which will be effective, it is generally necessary to reseal every four or five years. A certain amount of the rock chips will kick off under winter traffic and many of the seals tend to flush up and bleed during the first hot spring weather each year due to the release of film moisture or some unknown reason. As each reseal is placed, the ratio of asphalt to aggregate becomes higher and the flushing appears to occur at an earlier date as the excess asphalt accumulates. You do get in a vicious circle where you have to seal for non-skid earlier each time due to the flushing action, especially with some of the softer asphalt.

Mr. Nevitt: Sealing operations may perhaps defeat their purpose of a non-skid surface. Personally, I believe that can be cured by proper construction and material control. I believe it is possible to build seal coats that will not flush to the extent of ruining the original purpose, and the need then for further sealing is due to the need for new, non-skid aggregate, rather than excessive flushing.

L. L. Marsh:
Kansas

I may be out of order in bringing this up, but Mr. Hallvik mentioned a minute ago about sealing after a recently constructed mat or surface treatment. I want to find out a little bit about the accounting methods.

In Kansas we go out and construct a mat or surface treatment, and we know when we plan it that it should be sealed, but for some reason or other it is delayed. The next thing that happens is that it has to be sealed, and it becomes a maintenance charge, which I don't like. I think it is properly a construction charge if it is designed that way. I would like to hear from somebody in the house about that.

Mr. Nevitt: I think that is a very sore subject. I think in the western states, because the Bureau participates in construction, sealing is automatically going to be a construction procedure. Delay or no delay, it is going to be charged to construction, though later sealing is maintenance. Where you have state funds used there seems to be variation. What comment do we have on it?

D. L. Cheney: Montana
That is a question I would like to ask, too. We have been trying to figure out for quite some time how we can delay the sealing and still hold to the contract. Every year we finish our jobs the first of September, regardless of when we let them. It is too late to seal them. If we release the contractor and close the job we lose the Federal Aid on the seal coat.

If anybody has a solution on how to hold the job over to next spring and still get Federal participation in the seal coat and also get along with the contractors, I would like to hear about it.

Mr. Nevitt: I think that would be a very proper reply, if anybody can offer a solution. Everybody seems agreed that we ought to delay on seal coating, but where it is a Federal project we seem to be faced with the necessity of doing it right after the construction, and it universally works out that way. I guess the contractors have good reasons for that, regardless of when the job is put down.

We might have some resolutions at the end of the meeting. One is that the sheep wagon manufacturers please start making them with wider tires; another, that the Bureau participate in a seal as a separate contract next year.

R. M. Schwegler: Oregon
What is the contractor's reaction to delay?

Mr. Nevitt: That is a very fair question. I looked around to see if A. B. Nuss was here, to ask what he thinks about delayed sealing. Are there any contractors who would like to comment on the contractor's point of view on delayed sealing?

C. F. Seifried: Wyoming
We have gone through that angle, so we can lend a sympathetic ear. We all realize, regardless of the Bureau of Roads or anybody else, that we have to have seals. We have the contractors fairly well trained on the mats. We found on several jobs that if we can hold the job open with the Bureau of Roads and finance it, we can put the seal coat in at a later date, even delaying until the following spring. The accountants don't seem to care, if we keep the job open and don't close it, but it involves a little financing by the State during that season.

Certainly, where the Federal Government or the State pays for this seal, it seems to me the first seal is a construction charge. If we can't nick the Bureau of Roads for it and we have to do it in the spring, our practice is to charge that to construction. No matter who does it, it is work done, regardless of whether the maintenance forces do it or the contractor does it, and we charge it to construction.

A man came to me a month or so ago and he said, ''They sealed that job in the fall. It shouldn't really have been sealed, but they had to put it on. They only put on an eighth of a gallon. It really needed one-quarter of a gallon. This was two or three years ago. Now we have to go down and put an eighth of a gallon more on that particular job. I think that should be a construction charge, because they should have put a quarter gallon on in the beginning. They shouldn't charge the extra one-eighth gallon to maintenance.''

Mr. Nevitt: He undoubtedly had a low maintenance cost. I suspect, from all I have heard, everybody agrees the first seal should be a construction charge. That doesn't mean it necessarily is on the state or counties.

Is there anybody who disagrees with that? Are there any construction men who do feel that is a maintenance charge? The theory I had was that you hadn't finished the job until you had it sealed. You have to have the job reasonably impervious before you can say it is ready to make a showing on standing up, and therefore it is not finished construction.

Are there any other comments on seal coating?

C. F. Seifried:
Wyoming

Let me say one word to sum up one or two points you brought up. There are two functions of the seal coat. In the first two days of our discussions here, it was brought out by the technicians and the research men that asphalts do harden, they do oxidize, that they do do various other things, and that they do deteriorate.

We have to keep the air from those poor asphalts. One of the primary functions of the seal coat seems to be to keep the air from these asphalts which, as we were told here, do not hold up. Really, that one function, it seems to me, should be in addition to the waterproofing, to keep the air from the asphalts which deteriorate with air. That should be the one principal function, but perhaps under a subheading of that it is necessary to add new material to liven up, if you will, to add strength to the thin layer of asphalt which oxidizes or hardens, or whatever happens to it on the surface. So the function of that is to keep restoring the strength of the thin film on top which does wear off, on which there is a little difference of opinion.

Anything wears out. The whole power of the mechanism of the automobile—I wish there was a college professor to say this for me—is transmitted through the contact of the tires with the pavement, and something has to wear. When two bodies come together, they both wear. A pavement doesn't wear as much as the tires. There is a certain amount of wear on both of the surfaces. That has to be renewed, and seal coating is perhaps a temporary expedient rather than a permanent expedient.

Mr. Nevitt: We are getting a nice development of the purposes of seal coating. There is a big change in the past five or six years in the attitude toward it. You would be surprised to know that in a few states and districts practically no sealing is done. The job is built and there is a tendency to leave it, the effort being to build it initially so that it never needs sealing, but Mr. Seifried gives a pretty strong case for sealing, regardless of the excellence of the original construction.

If there is no further discussion of seal coating, I think we had better take a few minutes' recess, and then we will get into one or two things of other natures on which some of you may have questions.

I want to say it seems to me we are doing fine and getting lots of nice discussion.

We have had some good discussion on seal coating. One point was not made which I thought it desirable to bring up. We are talking about sealing without cover. The engineers who are doing that are doing it judiciously. In other words, the Conference should not get the idea that you simply put on seal coat material regardless of the condition of the road. Sealing without cover is only done where the texture of the road surface, the type of material, and so on, permit the adoption of the practice without destroying the non-skid effect of the road. That is done and can be done, but we only seal without cover where we can seal without cover safely, so don't get the idea that any road can be sealed without cover, because it certainly cannot. You have a dangerous situation in many types of surfaces.

I thought that was worth while to emphasize, that sealing without cover is only adapted to certain surfaces and certain types of procedure, rather than a general practice suitable for all jobs.

We have talked about the surface, and some construction points. One question was asked in this list of questions which I thought was rather fundamental, which I haven't heard discussed in any recent meeting. In fact, I don't see that it has been given a lot of consideration.

You have noticed in all the discussions the trend toward the heavier asphalts, particularly among the SC users the trend toward SC-4 or SC-6, depending on just what those viscosities are, but a definite trend toward the higher viscosity materials, and I think the same situation is true in the cutbacks.

We heard somebody say, "We have been using RC-2, but we intend to use RC-4." The question is, when we change to these heavier materials in our design, do we change the quantity of bitumen in the mix? I thought that was an excellent point, because the matter of bitumen content in mix design hasn't been brought up at all in this meeting.

I think it breaks down, first, into how you design. If you design by a laboratory test on the aggregate and the bitumen to be used—California is a good example, where they run their stabilometer tests on the mixture, the bitumen content is set by the actual mixture. If that is the case, of course you automatically take care of changes in the materials. You naturally use the new materials in your trials, and therefore compensate or adjust for them. I don't believe many states have that type of design. They are designing by formula, or some other method. Possibly those which design by some type of laboratory equipment may have some specifications, so to speak, which might be modifications in changing the type of bitumen.

I thought that was an excellent question to put before the meeting. Do you change your bitumen content? Do you change your design as you change to these heavier bitumens?

Can anybody get something on the record for the benefit of the man who asked this question, and sort of air out this matter of mixture design?

J. E. Buchanan:
Idaho

I think that question came up in the Fundamentals Section. While we were discussing that phase of it the other day, we passed over it and reserved it for this meeting. I believe Mr. Muir, or someone in Utah, has had experience in this matter. Is he here this morning?

Mr. Nevitt:

I haven't seen him. I was hoping he would be, but I haven't seen him.

J. E. Buchanan:
Idaho

I will presume to relate very briefly what he told me. In using the heavier SC's, it was his general impression that the quantity of asphalt, on a percentage basis, for instance, was a little higher than for the lower viscosity SC's.

Mr. Nevitt: Unquestionably, the literature indicates that the average writer on the subject recommends increased quantities for heavier bitumens, although most of it has been in connection with SC's. However, what the literature suggests and what the states do doesn't always coincide, so I would like to have some other comment on it.

T. R. Perry:
Iowa

On the inverted penetration type of surface, we find there is a considerable difference in the amounts of bitumen which can be used. For instance, we started out with .6 gallon of bitumen and 60 pounds of aggregate, applied in two applications. This rate worked out very nicely for the MC's and the RC's. We found that 0.6 gallon is too heavy, or too much for the SC's, so we either had to change the oil content or the amount of aggregate. We settled on the 60 pounds of aggregate and reduced the amount of SC down to about .48 gallon which seems to be about the correct amount.

Mr. Nevitt: As I get Mr. Perry's discussion, it is more a change in the type of product, rather than the grade in that type. That is another little different phase, and I think a very important one.

Mr. Hveem and I have had a long correspondence about this matter, and I have been trying to get information from him. He started off by trying to get it from me, but I think he knows a lot more about it, so I tried to get it from him. He reports there is very little to go on in changing the type of material, changing from SC's to MC's, and changing from SC's to RC's. That is another phase which I think is also a good point for discussion, both the effect of changing the grade, and the effect of changing the type.

V. A. Endersby:
California.

I would like to find out how far Hveem's principle of adapting the oil content to the surface area of aggregate has generally been practiced in the different states.

Mr. Nevitt: That is a good question. In other words, the California formula method of design is essentially what you are requesting, where a design formula is followed.

Clif Hallvik:
Idaho

We are building an SC-6 job. We didn't have any SC-6 on hand, so when working for quantities of oil, used the SC-3 or 4 that we had on hand. We sent to a refinery and got some SC-6 and ran some stability tests on the material. The stabilities went way down with equal amounts of asphalt. I contend that heavier asphalt with the type of material we had, which was possibly, at the most, 20% crushed, would reduce stability, and there is some question in my mind as to the length of curing time to the ordinary run of the Hveem test. It seems as though our molded specimen, if we let it set a couple of days, gets almost as hard as pavement, and we get high stability. Running at an ordinary test time, we wouldn't get the stability we expected.

We use the Hveem stability apparatus for determining our asphalt contents for mix designs, but it is not mandatory that the engineer use our figures. It is just a guide for setting up the quantities for the project, and a guide for the engineer to start his oiling operations.

Mr. Nevitt: Do we have some more comment? I think it is an interesting point to take up first. Are many people using the California design formula?

J. G. Schaub: Michigan

Our practice is entirely plant mix oil aggregate. We use an adaptation of the Stanton formula which our laboratory technicians have worked out, and which has some constants to take into account the absorbable qualities of the aggregate. We adjust, of course, for the character of the fines and base our oil content on our findings. We have some aggregates that take as low as 3 per cent by weight, and others that require up to 4 and 4½.

Mr. Nevitt: Does your formula take into account the change in capacity or other characteristics of the bitumen used? Do you always use practically the same viscosity?

J. G. Schaub: Michigan

We have been using SC-4 almost entirely, but this year laid two projects with SC-6, in which our mix design called for a slightly higher amount of oil. We also adjust for the specific gravity of the aggregate in our formula.

Mr. Nevitt: May we have some more comment on this subject? That is, another state that takes care of the specific gravity of the oil.

J. E. Buchanan: Idaho

I would like to know for sure if we are all talking about the same material when we say SC-6. The SC-6 in the Western region refers to a very heavy product formerly known as 90 to 95% road oil. In terms of penetration at 77 degrees, it would run above 200, possibly from 200 to 350. Is that the same SC-6 you had in mind?

J. G. Schaub: Michigan

I don't recall exactly, but the SC-4 and the SC-6 were the old designation.

Mr. Nevitt: Mr. Buchanan brought out an excellent point, and one that perhaps I am qualified to say a little on because I am a member of the asphalt technical committee.

The California SC-6 oil is a long way different from the Mid-continent SC-6 negative spot road oil. The two may have the same viscosity at the specified temperatures. In the new Asphalt Institute specifications, say, they will both be specified at 180, but at 77 they will be different materials entirely. From some sources the California SC-6 will show a penetration of from 250 to 300. The Mid-continent negative spot road oil at 77 will be a long, long way from a penetration asphalt. It will be quite soft, liquid, and have quite a little lubrication effect, just talking in a general way.

I have tried not to get too much into that, and let the record show SC-6 or SC-4, but I think it is a very worthwhile point that when somebody says he used a certain SC-6, or the SC-6 in a certain way, you should remember it may not be the same product at all, either due to the difference in the nomenclature, the difference in the time or specifications from year to year applying to that nomenclature, or the difference in the origin of the asphalt.

Can we have some more discussion of this point?

V. A. Endersby: California — What I had in mind was on the matter of the curing time of test specimens. I think you are working on something else still, aren't you?

Mr. Nevitt: The point was made that curing time is very important. Mr. Hallvik brought out that he doesn't get the same result. I think that is pertinent to this general discussion of mix design from the practical standpoint. I don't want to get too far into the theory. The Conference is supposed to present theory for the practical man, rather than a discussion between theoretical men. I believe it has been the general wish of the officers to let the practical men learn the application of the theory. However, we do have to get into a little bit of theory. The practical point of what we are actually doing is what I am trying to talk about. Are we changing the bitumen content? We have had some discussion on this phase.

In watching this, are we having to look out for these changes in grades, maybe from the laboratory standpoint? Are we having to watch the practical laboratory lining up of the job?

I would like to make a few comments myself, not as chairman. Mr. Hallvik, you report, as I gather, that raising the viscosity of the SC-6 actually tended to decrease the stability, whereas the usual thought is that it increases the stability. I think that is due to something that is being realized—Hveem has emphasized it to me quite a little

—that we get a certain amount of setting up, in the SC's particularly, by absorption of the light fractions in the aggregate. If you happen to have that situation and you get a very heavy viscous material, which has had all those real light, thin fractions removed by refining, as may be the case, you may get less of that absorption, so that you actually have thicker films and may overfill the voids. In other words, if you have a type of aggregate where overfilling the voids may be a factor, and you go to a thicker, more viscous material, you might not be able to increase the bitumen content, either due to the fact that you don't have the void space for it, or due to the fact that you don't get as much absorption in the heavier, more viscous material.

On the other hand, if you have an aggregate with high void space, I think you will find the general trend of opinion, that you increase your stability by the heavier materials and need more bitumen, is borne out by your actual experience. So we have to interpret some of these things we find.

Clif Hallvik:
Idaho

That is the conclusion we arrived at more or less in the laboratory, that we were getting much thicker films. The question that enters my mind is this: Isn't it perhaps better, in setting up our SC-6, as we have it in the West, to construct oil mats with entirely crushed material rather than, as I stated in my previous remarks, between 15 and, at the most, 20 per cent crushed material? If we had 50 per cent, or better, crushed material, I think we would get better stability and a better mat with the heavier grade of asphalt.

Mr. Nevitt:

You are getting into the point brought up the other day about round versus sharp aggregate, and your thought is if you are increasing the bitumen content it does tend to lower your stability, but you can compensate for that by crushing. I think that is unquestionably true.

There is another factor in crushing. You generally increase your void content, so you have less chance of over-filling your aggregate to get these thicker films that most of the asphalt technologists agree are better from a life standpoint, if we can get them and don't have trouble from getting them.

Clif Hallvik:
Idaho

I would like to hear Mr. Endersby's remarks on the curing time on test specimens.

V. A. Endersby:
California

Two or three different factors operate to produce greater stability. One, of course, is the evaporation of the solvents they use in RC's and SC's. There are also some chemical effects in the residues that take several days and sometimes several months to develop, in addition to the absorption in the aggregate, and so on. We haven't any definite figures on how long samples should be cured, but we think as short a time as 24 hours is altogether too short to give any good representation. A week, perhaps, would be better.

Mr. Nevitt: On that 24 hours you are now using, but which you think is too short, are you doing that at room temperatures or at elevated temperatures?

V. A. Endersby: California.
Room temperature.

Mr. Nevitt: Do you not believe that a shorter time at an elevated temperature, to permit speed of the volatilization, absorption, and other characteristics, may offset that?

V. A. Endersby: California
It would change the volatilization and might also change the other characteristics, but we haven't any definite data on that.

Mr. Nevitt: I am talking a lot about Hveem, because they are doing so much out there. I suppose you Western fellows know that Hveem is putting in some sort of curing apparatus, a rather elaborate mechanism, which heats the aggregate mixture, cools it, and puts it through a series of operations by which he is trying to get this laboratory curing, and accelerated curing incidentally, which will give him a mix that corresponds to the road after a reasonable time. He plans to get that and correlate it with service.

J. E. Buchanan: Idaho
I would like to ask Mr. Endersby if during the 24 hour period it is a loose mixture or a briquette.

V. A. Endersby: California
Briquette.

H. F. Winterkorn: Missouri
You were just making a remark about increasing the speed of reactions. If you really want to have chemical reactions going faster, the increase of temperature helps an awful lot. There is even a rule for most of the organic reactions, that the speed of reaction increases to the double or more by the rise of temperature of 10 Centigrades. Therefore, the increase of temperature will go a long way in making the curing time shorter.

Mr. Nevitt: I think we are going to have to do something to accelerate the curing. We have to cure more or less, as brought out by Mr. Hallvik. I think that has been everybody's experience. We have to do it in a reasonable time to get practical results. Temperature looks like one way to do it.

W. S. Housel: Michigan
I would like to ask some of the men here what their thought is on the maximum temperatures that should

be used in laboratory curing. That has been rather a sore point in setting up some of the research projects, particularly in airport construction where we want to cure some of the specimens by setting up definite conditions during the curing period. I think 140 degrees Fahrenheit may be satisfactory on the basis that it is the generally accepted maximum pavement temperature. I am wondering what the other men, who are having some experience in the laboratory in that respect, think about the temperature we should use in curing.

Mr. Nevitt: Has anybody any comment on the proper temperature to use in the laboratory curing? We don't want to get too far away, but that is a pretty fair question because all the laboratories are going to get into that for practical design, I expect.

We are using 140 degrees, and while we haven't gone far enough to know, we begin to get a little evidence that I think is going to indicate it won't take an excessive time to get equilibrium, whatever that may mean.

W. S. Housel:
Michigan

I judge, in the absence of any definite remarks to the contrary, that the idea of not going above possible high temperatures in the pavement is a safe thing to accept. If you go above that, you don't know when you are getting into something that doesn't conform to conditions in practice.

Mr. Nevitt: It would seem 140 was not too high, because pavements do reach that temperature practically every summer. The question is whether 140 is too low, rather than too high. Our slight work to date indicates it isn't too low and you won't have to wait an excessive period to get some sort of equilibrium, if you define equilibrium as exhibiting the same characteristics as you continue the curing. We don't know yet, but it seems as though that will be all right.

We don't want to get too far away in the theory. We have had so much research and fundamentals that I want to stay down on the practical application.

We have talked over mix design quite a little. Does anybody want to bring up any more practical problems on that? Are there any other problems on the bituminous mat that somebody here would like to ask? We have lots of questions, and the thing to do is to get the questions that most interest the people here.

Art Loforth:
Nevada

Mr. Nuss brought up a point the other day about noon, but seeing that everybody was so hungry it couldn't quite soak in. I think it is of more concern to Mr. Nuss now that he is a contractor than when he was chief engineer of a typical prairie state, and that is the matter of moisture in windrow. How long should you mix it to get the moisture out, and what is a positive test for the amount of moisture allowed?

I can recall one case in Nevada, in 1929. We were road mixing in the early part of the year. We mixed a section for maybe three or four days, off and on. The last day we got a good heavy shower, and the foreman said, ''We will lay it down and tear it up the Fourth of July.'' The fact is he laid it down and it has never been torn up. Maybe the secret of that success was the fact that in those days we didn't apply any seal coat and the moisture in there might have evaporated.

I can see Mr. Nuss' point. It must be very disconcerting to have some inspector tell him, ''Mix her out; we want the moisture out,'' and maybe he doesn't know, and we don't know just where to draw that line on the amount of moisture permissible.

Mr. Nevitt: I agree with Mr. Loforth. When that was brought up I felt it needed a great deal more discussion than it had then. I wondered if anybody would be interested enough to bring it up today. I suspect it is going to turn out as on this seal coating, where you do different things for different reasons. I think the session here might bring out the controlling factor. I know I had a couple of thoughts on it, and I have no doubt there are plenty of others. Let's have some discussion on that.

Does anybody have any definite ideas or recommendations? You all have specifications, so you either ought to defend them or get up and say why you think you are willing to change them. Do we have some experience?

Here is one case of a mat mixed wet, and no trouble. I have no doubt there are are more of that type. Are there also experiences of the mat being mixed wet, not too wet, not really an excessive amount of moisture, and our usual water trouble showing up?

W. S. Housel:
Michigan

I may be wrong, but I can't help but feel that letting down the bars on moisture content is what is spoken of as wishful thinking. It seems to me that in all of the instances that have been brought up, like the one Mr. Loforth just cited, where high moisture content has not been detrimental, almost invariably involve a compensating factor in the question of moisture content. The surfaces were not sealed immediately and there was a chance for the moisture to get out. It seems to me, in the case Mr. Nuss talked about the other day, that was also true.

As far as the character of the test specimens that we make in the laboratory are concerned, both in our soil stabilized mixtures and in the bituminous mixtures, it seems to me very clear that the moisture has to be gotten out of the mixture sometime before the road is subjected to a long period of service.

In that connection, it seems to me there was quite a comprehensive investigation made by the Bureau of Public Roads in Wyoming—I think in Wyoming, wasn't it—several years ago, and as I recall it the statement was very positive and very definite that any roads that had been laid with over 2 per cent moisture content had given very bad results, and that the failures on those that had been put down with less than that were the exception rather than the rule.

I have seen no evidence to lead me to accept any other conclusion with respect to bituminous mixtures, and it seems to me that when we get up in a conference like this and discuss the cases where excessive moisture has not been detrimental, we should be very careful to ascertain whether or not that moisture had a chance to get out before the road was really subjected to service before we reach any conclusions that moisture isn't harmful.

C. E. Remington: Montana

The contractor isn't particularly interested in the amount of water that is in the mix when it is laid. He is interested in how little water he can have in it when he starts to put the oil in the mix. I think we have all proved in the field that the mix for the mat may be wet when you start to put your oil in it, but that you can drive out a considerable proportion of that moisture just in the mixing out of the aggregate itself. This pet job of mine, right here in town, was laid wet. The oil was put in it when it was wet. Being close to the hills, if we had run a moisture test, it would probably have gotten wet again before we received the results of our test. So we have to use our judgment when we put our oil in it, and hope we will get rid of that moisture before we have to lay it. I think we do, because the mat ordinarily has been a success, even though the oil was put in when the aggregate in some instances was very wet.

Guy J. Johnson: Montana

I think one would have to be very choosey on the aggregates he allowed to be oiled while wet, because we have some aggregates which are entirely satisfactory if oiled dry, but on which the bitumen will emulsify if applied while wet. This particular type of emulsification is not a beneficient one; it is a destructive one.

H. F. Winterkorn
Missouri

Mr. Chairman, I want to refer to some of the remarks I made in the Fundamentals Section, or some other time, about the moisture in relation to stripping which, of course, is related to this general problem. There, it appears that we might allow different amount of moisture for different stones.

It all goes back to the tenacity with which the stone holds the moisture, and how much it can hold at high temperatures.

There may be a very interesting parallel in the bituminous soil stabilization. When we permit a mixture of soil, water and bitumen to dry out in the laboratory, then we find that the resistance to re-entrance of water is more or less a function of the chemical properties of our soil materials. But if we don't allow it to become entirely dry, like we have in some instances on the road where we mixed rather wet and where the stabilized mixture never really dried out, we found that the amount of moisture in that road after a year of service was proportional to the plastic index of the material—was proportional to the water-holding capacity of the material. If you permit moisture to stay in there, you might not for a long time have a reaction between the bitumen and mineral, which is necessary to give a stable system. We have a moisture film in there which keeps bitumen and mineral apart.

R. M. Schwegler:
Oregon

I think we are getting on rather a dangerous topic when we try to make a general statement of increasing the allowable amount of moisture we can leave in the aggregate. There are, undoubtedly, aggregates that can be laid fairly wet without any detrimental results. On the other hand, there are aggregates where we get into a lot of trouble with excessive moisture.

In my opinion, if you add 5 per cent of moisture to a mix, and lay it and seal it, you get the same result as if you had added an additional 5 per cent of oil to the mixture; all your voids would be filled and you would lose stability. I believe the practice is very dangerous, especially when we are laying our mats late in the fall, due to Federal Aid practice, and sealing immediately. I think we had better play safe and keep the moisture out.

V. A. Endersby:
California

One point that might be of some importance, is that if you allow a bituminous material to take up water, dry out, pick up water again, as time goes on it takes up water more easily the more often it has been wetted.

Mr. Nevitt: That is an interesting point.

H. F. Winterkorn: Missouri — That again is dependent upon the composition of our mineral aggregates, because some aggregates, after having been treated and cured with bitumen, after continuous immersion will take water more easily after awhile; others may not take it more easily. So it is a function of the mineral again and, of course, to a certain extent, of the bitumen.

Mr. Nevitt: I used to get a lot of ideas about water and mixes from the Idaho boys I talked to then, and I thought they had a lot of experience with water. Mr. Flint, have you any comment on this matter?

H. R. Flint: Idaho — Mr. Chairman, I think water is a good thing. I would prefer that our materials men, and men who are out in the districts, discuss that, please.

Mr. Nevitt: If you have any comments, we would like to have them. There were some definite opinions from Idaho.

Clif Hallvik: Idaho — On our road mix work we have never worried a great deal about what the percentage of moisture was. It is just a question of whether it looks dry. We didn't take any tests. Most of our roads have been quite satisfactory. Of course, when we bump into a rainy spell, it is a general rule that the road will go to pieces. So I think the inclusion of water in the mix, as the gentleman here said, is dangerous, because sometimes we point to a project that was successful and forget maybe a dozen that weren't so successful.

Mr. Nevitt: I might interject one point. Apparently the situation boils down to this: That we may stand a lot of water, but we might have emulsification on some jobs and have trouble there. We might have a mineral aggregate which has some of the tendencies we have classified under the head of stripping, which may cause trouble. I have become more interested in stripping from that angle than I have from any other. I believe the speed with which the moisture is displaced—because there is always some moisture—is rather important in your construction.

There is another angle on which I should like to have comment. I have had an idea which may be entirely wrong, that the cutback jobs showed much less water trouble, and have thought the reason for it might be the tendency of the volatile in leaving the mix to carry moisture with it, by the usual partial pressure effects we have in distillation. I believe I am right on the practical conclusion that we have less moisture trouble with cutback jobs—not universally eliminating, but decreasing it. Maybe my theory that the volatile helps carry off the moisture is correct.

Does anybody want to comment on that?

Guy J. Johnson:
Montana

There, again, I think we are up against our mineral aggregate. We have probably not gone far enough along that line to have much worth commenting on, but we do have definite types of aggregate where slow curing oils will strip less readily than MC cutbacks and RC's, and conversely. It is largely a matter, in our opinion to date, of fitting the type of asphalt to the aggregates.

V. A. Endersby:
California

I was wondering if it wasn't a matter of how liquid the material is when you mix it. The more liquid it is, the more easily it will displace water from the surface of the aggregate. I don't know whether anybody has any data on how far that is true.

Mr. Nevitt:

That may be a reason, although my personal opinion is that a cutback of the same viscosity is less susceptible to water than an SC. Do we have any more comment.

I judge the feeling of the meeting is that you might raise this water limit, but you might get in trouble if you did it in a considerable proportion of your jobs. So, to play safe, you had better hold it down as far as you reasonably can, that being a matter of practical judgment. This conclusion probably represents our present judgment as to the desirable situation.

H. R. Flint:
Idaho

I did run on to one example that I would like to comment on a little. It was a matter of the rock chips being placed. It was rather late in the year, and of course our thought was that the chips really should be taken through a drier. We had no drier available and there had been considerable moisture. Actually, when we took those chips up to prepare for the spread, free water ran out of the bottom of that truck. I expected complete failure, but it turned out to be an excellent job.

Mr. Nevitt:

We have just a short time left. I would like to get any subjects, preferably those we sort of cut off in mid-air, as we did this one; or there may be some others that somebody has a feeling are rather urgent and he wants to get some help before he leaves the meeting.

I didn't mean to cut Mr. Keeling off before. I know you had something.

R. C. Keeling:
Kansas

I was going to ask what the different states are using for rolling in the cover material in the seal coats. The general opinion seems to be that if a person would go to the pneumatic-tired roller, he will imbed the material in the seal coat better than he will with a smooth-faced roller. We have been using smooth-faced rollers, and there is a question in my mind whether we ought to go to the pneumatic-tired roller.

Mr. Nevitt:

With the development in seal coating, we ought to get some refinements and improvements in practice. Here is a possible one. How about it? Lots of states are doing seal coating now and rolling in chips.

R. C. Keeling:
Kansas

I think Texas demands a rubber-tired roller.

Mr. Nevitt:

This is a little out of my line. Isn't there a general trend to pneumatic-tired rollers almost everywhere? Where both have been used, isn't there a swinging over to the pneumatic-tired roller? Does anybody have any direct comment on Mr. Keeling's question?

W. S. Housel:
Michigan

I don't have anything to offer with respect to seal coating. The experience we had was on a soil stabilized base and using the conventional roller, which gets the traction off the wheel doing the rolling, and in that particular case we developed a scale of about three-quarters of an inch or an inch deep. This curling and scaling, as nearly as we could tell, was due to the horizontal displacement of the mixture being compacted because of the traction being taken off the rollers. Very early on that job I insisted that we should have a rubber-tired roller. While we didn't get it on that job, I still think that pneumatic-tired rollers may have eliminated the difficulty.

This matter of the horizontal thrust is something I am glad to see get more attention. It is surprising how heavy the thrust force is in many cases.

J. E. Buchanan:
Idaho

This may be appropriate to Mr. Keeling's question. In reviewing some 15 or 20 papers for the Fundamentals Section, I recall that a number of the individual committee members referred to the increasing use of rubber-tired rollers. That wasn't included in the summary of the Fundamentals Committee because I thought that belonged to construction, but as long as the comment was made and I observed it in reviewing the papers, it bears on your particular question.

Mr. Nevitt:

We have a few minutes. Here is your last chance if some of you fellows have some things you want to get off your chest.

V. G. Gould:
Iowa

In connection with the pneumatics versus steel rollers, which I believe Mr. Keeling's question involves, we have had more experience in this connection with surface treatment than with seal coats. I see no reason why the two operations are not comparable. At the present time our conclusions are that the type of oil being used has a direct bearing on the type of roller preferred. Assuming that what you are trying to get is the maximum amount of cover aggregate imbedded in the binder material at the minimum cost or as quickly as possible, it doesn't seem to make any particular difference what type of roller is used with SC oils because no film forms for some time and aggregate will imbed quickly from pressure. However, in the case of cutbacks which form a film rather quickly, the pneumatic tire is not as effective in imbedding the aggregate as a steel surface.

Perhaps later we may change our opinion but at the present time these statements appear correct.

Mr. Nevitt: Are you talking about chip seals or inverted penetration treatment?

V. G. Gould:
Iowa

Inverted penetration, but I suggested the same principles might apply to the seal coat.

Mr. Nevitt: There is a little difference, don't you think? You have a little thinner layer, a little harder surface to work on, and so on.

I think one thing we have brought out here is that seal coating seems to be quite generally adopted. The reasons for it have begun to be appreciated more widely, and there is more than just one reason.

The next thing is that we don't seem quite to have arrived at the ideal seal coat, particularly where we have these cover seals. In other words, progress still needs to be made. We apparently need a sealing method with cover chips which doesn't bleed up through, and probably a little more knowledge of the construction practice to be sure we have the best.

So we have apparently brought out quite a little on sealing and what we need to look forward to, which I think is always helpful. Lots of times you know you ought to be going somewhere, but you don't know what you need to do next.

C. E. Remington:
Montana

There is another point in connection with rolling with a steel-wheel roller that you have to watch, and that is rolling when your surface is wet, or rolling with wheels that have been treated with water. That will cause the flaking that Housel was talking about. Our specifications allow something applied to the roller wheels to prevent their picking up the mix. That usually allows the use of water. We have found considerable trouble using a steel roller with water on the wheels, because there is flaking of the surface caused by forcing the water into the surface under the wheels of the roller. You get away from that with the pneumatic tires.

V. A. Endersby:
California

Closely related to that is the fact that the steel roller puts on tremendous pressure, about 1000 pounds per square inch. A rubber-tired roller allows the material to adjust itself a little; the aggregate comes out from under the roller intact, where the steel-tired roller will break up the material, making bare surfaces more susceptible to water.

Mr. Nevitt: Quite different results apparently. Is there any more discussion on that point?

W. S. Housel:
Michigan

Mr. Chairman, there is an interesting experience I might bring up in connection with rolling. I was talking to Dr. Bernhardt who had considerable experience as an investigator in connection with engineering construction in Germany. He suggested that the method of rolling may be the source of the waves or corrugations in these so-called washboard roads. Referring to an investigation they made over there on steel rails, while it seems rather widely separated from what we are talking about, they found that the rolling of the rail produced irregularities in its structure which, due to abrasion after the rails have been in service for some time, led to uniform irregularity and gave them a great deal of difficulty.

He had the idea that possibly we were getting some of this washboarding in the bases that were being rolled from the fact that the roller, particularly at the heavy pressures which Mr. Endersby mentioned, would tend to produce successive waves of displacement and the weaknesses thus introduced were the beginning of the instability which produces washboard roads. This condition appears to be particularly applicable to the usual steel rollers.

I had always supposed that the washboard effect in gravel roads was related to the vibration frequency of the wheels, or something of that kind, but they tell me there has been considerable investigation by the vibration experts and they can find no correlation between periods of frequency of axles and wheels, and these washboard bumps that come with surprising regularity.

I just put that point forth as something to think about in connection with the discussion we are having right now. I have no opinion on it myself.

Mr. Nevitt: Very interesting information. Do we have any more comment, or last-minute short questions?

R. J. Templeton:
Wyoming

Two years ago we had a five-mile project which had originally been surfaced with Maury shale. It became loose and we wished to put on a base gravel course, and then a mat. We attempted to do the rolling of the Maury shale with a steel-tired roller. We had the experience Professor Housel mentioned. It flaked and shoved. We brought in a rubber-tired roller and it surely did the job fine. We got the kneading action which is so desirable. With the proper amount of moisture, we got a hard surface and put on the additional base course and had no trouble.

Mr. Nevitt: Rubber-tired rollers seem to have great possibilities. At the Omaha airport I noticed when the mat didn't look so healthy, they used a rubber-tired roller, and rolled it back and forth to get compaction and kneading action to give the mat some life. It looks as though it has possibilities.

Is there any more discussion of this?

Gentlemen, I think you have shown very commendable fortitude. Our time is about up, and I don't think anybody would object to having it shortened about two or three minutes rather than lengthened an indeterminate period.

H. R. Flint:
Idaho

I have no announcement to make; I don't know that it is necessary, but I do wish to say a few words.

As official spokesman for Idaho, I just want to express our gratitude to Mr. McKinnon, Mr. Cheney, and the other representatives for coming to Idaho and helping us to share some of the glory of such a convention as this.

Our work, as far as we are concerned, has been handled almost exclusively by Mr. Walther, whom you have probably had little opportunity to meet because he has been so busy. He has been assisted by others, such as Mr. Hallvik, Mr. Bob Jewel, and they have borne Idaho's burden in this thing.

We do hope you have enjoyed it. It seems as though it has been a very instructive and helpful Conference. We hope you will feel kindly toward Idaho, and we will be very happy, some time at your convenience, to have you back with us again. Thank you.

Mr. Nevitt: Gentlemen, I wish to take the opportunity to tell Mr. Flint that we have certainly appreciated the hospitality of the Idaho people. He merely needs to look at the delegates to see that they have enjoyed it and hear what they are going to say about it in later years. So we can adjourn on the basis of thanking the Idaho people for their hospitality.

The meeting is adjourned.